CLEAN-COAL TECHNOLOGY BY-PRODUCTS USED IN A HIGHWAY EMBANKMENT STABILIZATION DEMONSTRATION PROJECT

A Thesis

Presented in Partial Fulfillment of the Requirements
for the Degree Master of Sciences in the
Graduate School of The Ohio State University

by

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MASTER'S THESIS ABSTRACT

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THESIS TITLE: Clean-Coal Technology By-Products Used in a Highway Embankment Stabilization Demonstration Project.

Summarize in the space below the purpose and principal conclusions of your thesis. Please single space and do not exceed 100 words.

Clean-coal technology by-products are used in a highway embankment demonstration project. This research chronicles the procedures used in the process and analyzes the stability of a repaired highway embankment. The reconstructed slope is analyzed using an Intelligent Discussion Support System that was developed from a slope stability program. Water quality studies are performed and an instrumentation plan is suggested.

The calculated factors of safety and the observed embankment performance give indications that the field demonstration project was a success. Long-term monitoring will be the best barometer for determining embankment gross movement and the future of FGD by-products as a stabilizing material.

William E Wolfe

Advisor's Signature

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CHAPTER I.

INTRODUCTION

1.1 Background

The combustion of coal containing sulfur in U.S. power plants is thought to be one of the principal causes of acid rain in North America. The Clean Air Act of 1970 was passed as an attempt to reduce the environmental threat from sulfur released into the atmosphere. In addition to numerous other standards, this law established a permissible level for emissions of sulfur dioxide (SO_2) from coal fired power plants. Amendments in 1977 and in 1990 to the Act have considerably strengthened its provisions with respect to the allowable levels of atmospheric SO_2. Subsequently, many power plants have either opted to purchase more expensive low sulfur coal or install desulfurization systems. There exist technological limitations to the complete switching to low sulfur coals, primarily lower heating values and differing fouling characteristics. The alternative, desulfurization systems, typically work by injecting reagents that combines with

desulfurization systems, typically work by injecting reagents that combines with the sulfur to form a solid compound which can be collected prior to the atmospheric release of the exhaust gas (Bigham et al., 1993).

The most commonly encountered methods of desulfurization involve either wet scrubbers or dry scrubbers. In power plants with wet scrubbers, which are more frequently used, the particles in the exhaust are removed and the gases are mixed with reagents in a slurry. The reaction of the SO_2 with the reagent creates a paste-like waste product which must be collected, dewatered, and eventually disposed of.

In the dry scrubber process, the reagents may be mixed with the coal at any number of stages along the combustion process. The reduction of gaseous oxides, as well as other pollutants outlined by the Clean Air Act and its Amendments, can occur during pre-combustion, combustion, or post-combustion stages. By removing pyrite and other mineral particles, pre-combustion coal cleansing reduces the non-combustible components that lead to SO_2 emissions. Another technology allows the SO_2 and NO_x gases and solid particles to be removed during the combustion process, while the post-combustion method removes the pollutants after the coal is consumed. A fourth way of reducing harmful emissions is through coal conversion, which is a process that alters the coal into a fuel consumed as either a gas or a liquid, thereby avoiding the conventional coal combustion process (Tismach, 1993).

The flue gas desulfurization (FGD) material used in this study was produced at American Electric Power's Pressurized Fluidized Bed Combustion (PFBC) Tidd plant. In a PFBC facility, cleaning is achieved by controlling combustion parameters, such as oxygen to fuel ratios, temperature, fuel feed rate, and by injecting sorbents directly into the combustion chamber (Bigham et al., 1993). The resulting solid waste product is collected and must be properly discarded. Current regulations treat the scrubber sludge from the desulfurization process as a solid waste and require that it be deposited in a controlled landfill. Utility landfill costs vary from as low as $12/metric ton to as much as $35/metric ton, with prices surely to increase as regulations get more stringent and existing landfills reach capacity (Wolfe and Beeghly, 1992).

The presence of free lime in the dry-FGD process, along with the inherent properties of the fly ash, are two of the reasons this material is being considered for beneficial uses. Other characteristics that make this waste product attractive are its extremely low cost and its good shear strength characteristics. The cost of using FGD by-products as an engineering material is directly related to the cost of transporting the waste to the desired location minus the cost associated with alternative uses - primarily land filling. Therefore, if the waste material can be delivered to the beneficial use location for the same cost as land filling, then material costs would essentially be zero. There are also numerous social benefits which are not considered in this simple economic analysis.

Unfortunately, the properties of the FGD material vary greatly as the procedures involved in production vary, such as the percent of lime added, the sulfur content of the parent material, and the specific scrubbing process. These variations often produce an inconsistent product on which the construction industry is reluctant to develop a reliance. Unknown long-term environmental effects from the FGD by-product also hinder the widespread acceptance of this class of material.

1.2 Objective

Because the volume of solid waste production is so great and because land filling is becoming a less attractive solution to the solid waste problem, various groups have been attempting to identify potential uses for the FGD by-products. In 1991, U. S. coal combustion facilities landfilled almost 18 million metric tons of FGD waste (American Coal Ash Association, 1992). Kentucky, which contributed over 2.5 million tons of FGD waste in 1991, expects that by the time all power plants comply with the Clean Air Act Amendments, the amount of waste will exceed 4 million tons annually (Hower and Rold, 1993). Using Kentucky as a model for additional FGD by-product production, the year 2001 would see an increase of over 50%, to nearly 30 million tons. As these numbers continue to grow, the urgency for increased use becomes apparent. Some of the more promising beneficial uses for this material include high volume applications such as structural fills for highway embankments and ramps, backfills for retaining walls, and as the select material used in subbases and base courses for roadways.

Three demonstration projects in which dry FGD waste is used as a construction material are briefly described in the following section. Chapters Two and Three describe in detail a field demonstration where clean-coal technology by-products were used in an embankment reconstruction project and the corresponding stability analysis. Several conclusions are drawn from the completed projects and recommendations are made for further studies.

Figure 1.1. Demonstration Project Site Locations.

1.3 Potential FGD By-Product Uses

To establish the types of applications for which FGD by-products are well suited, three field demonstration projects are outlined in this section. The main objective of this section is to demonstrate to the reader the wide range of beneficial uses that are possible with this material. The locations of the four projects that are to be described have been identified on a state of Ohio map shown as Figure 1.1.

1.3.1 Truck ramp

A truck ramp was designed by engineers in the Ohio State University Department of Physical Facilities to provide a location for the unloading of hard trash (Site #1 on Figure 1.1). The ramp, seen as Figure 1.2, was designed to be 17 meters long by 7.5 meters wide by 1.2 meters high. The by-product used in the construction of the ramp was a spray dryer material generated in the McCracken power plant on campus and was delivered to the construction site by contract haulers. The material was to be placed within 5% of the optimum moisture content (55%) and greater than 90% standard proctor density (0.91 g/cm^3). The ramp was constructed by University maintenance personnel during work schedule breaks in the summer of 1992, using only University owned equipment.

Tests performed on samples cored from the ramp over the first ten months after it was placed into service show that although the ash was not saturated, the water content was considerably higher than the optimum water content. The in-place

density was found to range between 0.83 g/cm^3 to 1.06 g/cm^3. Laboratory strengths obtained from unconfined compression tests performed on the cored field samples varied greatly (76 to 840 kPa) and were noticeably lower than the values achieved on the laboratory compacted samples. These variations in properties emphasize the importance of maintaining proper control over moisture and compaction during construction.

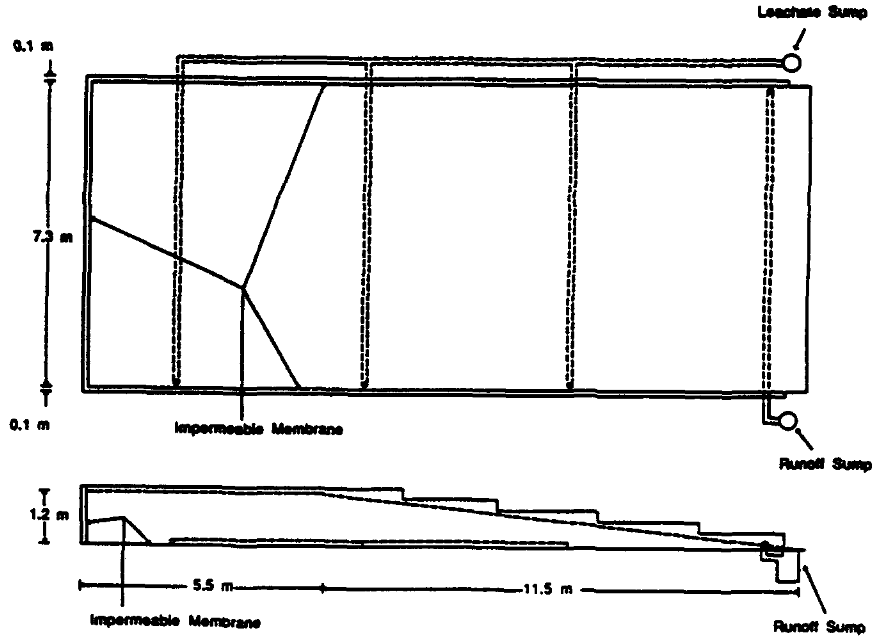

Figure 1.2. OSU FGD By-Product Truck Ramp, Plan and Section View.

Despite the difficulties in achieving the design conditions during construction, there have been no problems with performance and no evidence of distress. For the sake of brevity, the engineering properties of the OSU FGD by-product and typical engineering properties for soil were excluded, but can be found in Adams et al., 1992. Water samples have been periodically collected from both underdrains

and from surface runoff and have been analyzed for pH and metals content. The pH has remained in the region of 9 to 10 and measurements of metal concentration have been significantly lower than U.S. E.P.A. allowable limits. The results of a typical metals analysis are presented by Wolfe et al., 1992.

1.3.2 EORDC Bull Test Station

The Ohio State University operates a research farm (Eastern Ohio Resource Development Center) in Belle Valley, Ohio, which is shown as Site #2 on Figure 1.1. One of the activities at the station is a long running study that observes the effects of several factors on the growth rate of bulls. Twice a year, young bulls are brought to the center and raised in feed lots where their diets can be carefully controlled. The feedlots, which are not covered, have always had problems with too much water causing the bulls to sink into the saturated soil. A lack of stable footing causes the energy expended by the bulls to increase and weight gains are reduced. By stabilizing the feedlot floors, one of the uncontrolled variables in the test program would be removed. The growth of the animals would be improved, as would the reliability of the conclusions drawn from the EORDC studies. More importantly, identifying an inexpensive and reliable method for stabilizing feedlot floors could reduce substantially the cost of raising beef cattle in high rainfall areas such as Ohio.

In this demonstration project, dry cyclone ash from American Electric Power's PFBC Tidd plant was used to stabilize the saturated and organically fertile in-place

soil. This was done by blending the dry ash into the top 20 cm of the in-place soil and compacting the mixture to produce a stabilized base. Once each pen was treated, strength gains were fairly rapid. A cover of 20 to 30 cm of compacted dry FGD by-products was then placed over the stabilized base. All work was performed by EORDC personnel using standard farm equipment. Laboratory tests were conducted on samples of FGD material to determine optimum moisture and density levels, unconfined compressive strength, swell and consolidation according to the procedures specified by ASTM (1990). These results can be found in Bigham et al., 1993.

Presently, four cycles of test cattle have used the stabilized lots. After the first cycle, some minor failures were observed in two of the three lots. These were repaired before the second cycle of animals was brought in. Some minor spalling was found at the joint between the FGD by-product base and a concrete apron on which the feed bins are located. After only a few cycles, the conclusions to be drawn are only tentative, but it appears that the PFBC ash does reach high enough strengths to warrant serious consideration as a soil amendment or as a soil replacement, even in the harsh environment present at the EORDC feedlots.

1.3.3 Ohio State Route 83

Approximately 300 meters of SR 83 on a hillside in Cumberland, Ohio (Site #3 on Figure 1.1), has been damaged by landslides. This section of roadway has experienced settlements for a number of years, so after repeated patchwork efforts,

it was determined that a more substantial repair project would be undertaken. Surface conditions indicate that the current slide is rotational in nature with a scarp roughly following the centerline of the road. A subsurface investigation of the site was conducted in early September, 1993, by ODOT. Although the full results are not yet available, the base of the slide appears to be approximately 6 meters below the roadway. However, there is ample evidence that the area has experienced multiple slides in the past. This demonstration project calls for the Ohio Department of Transportation (ODOT) to excavate the natural soil above the slide plane and replace that soil with dry FGD by-products from AEP's Tidd plant. A sketch of the intended design for this site is given as Figure 1.3. The measurements made on the Tidd by-product at EORDC clearly show that when compacted, this material reaches a very high strength and would therefore be a suitable material for stabilizing this slide.

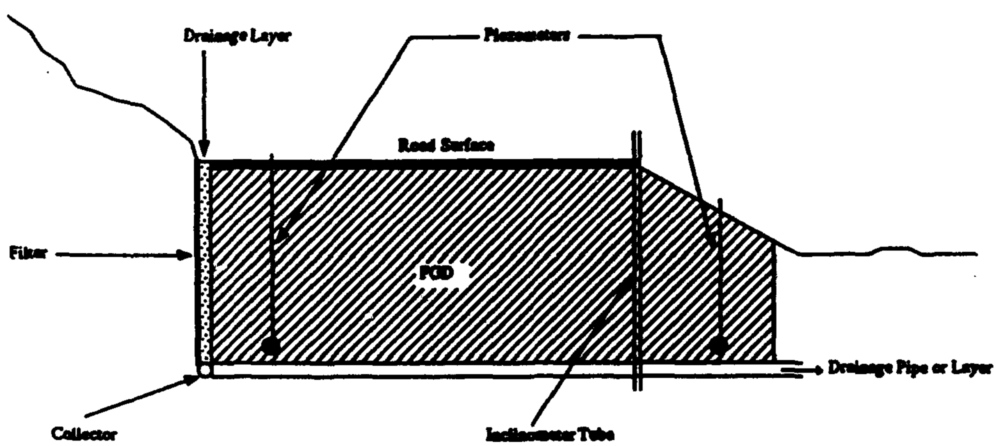

Figure 1.3: SR 83 Cross-Section.

1.4 Scope and Limitations

The disposal of flue gas desulfurization (FGD) by-products has become a major concern as issues of emission cleansing become more public and landfill costs continue to rise. Mixed with coal ash, the FGD by-products possess certain engineering properties which have been proven to be effective in a considerable number of construction uses. The purpose of this research is to document potential high volume uses for the FGD by-products. Specifically, the following chapters demonstrate that this class of material can be used with standard highway department equipment to stabilize an embankment relying only on routine repair procedures. The material does not require special handling nor does it have significant monetary costs associated with acquiring it. Our previous demonstration projects have shown that the FGD material performs well even though laboratory strength values may not be attained in the field, .

The remainder of this study is limited to a description of the behavior of one particular field demonstration project. The applicability of the findings to other projects will vary with material properties and geometric configurations. The intent of this analysis is to chronicle the events of an embankment reconstruction and show that the possibility for high volume use of clean-coal technology by-products does exist. The success of these experiments should lead to increased acceptance of this class of waste material in various construction projects. The

monetary savings will be realized in the reduced disposal costs for the waste, as well as the reduced reliance on alternative engineering materials.

CHAPTER II.

SR 541 EMBANKMENT: HISTORY AND PROJECT REPORT

2.1 State Route 541 - Site History

The portion of Ohio State Route 541 (SR 541) that is presently under study is identified on Figure 1.1 as Site #4. It is located west of Coshocton, Ohio, and is approximately 1000 meters long. It was designed in 1965 by the Ohio Department of Transportation and was constructed in 1966. Prior to 1966, the highway route was slightly further to the south and encompassed a series of sharp horizontal and vertical curves (ODOT Construction Plans, 1965). The SR 541 realignment was designed to facilitate high speed travel by eliminating the vehicular slowdowns that were associated with the original section of highway. To prevent steep vertical curves and to ensure adequate roadway drainage, the 1000 meter addition required the construction of a large embankment requiring a great deal of fill. The site map of this project is shown as Figure 2.1.

The design designations for this section of highway are displayed in Table 2.1. No information was attainable to confirm the Average Daily Traffic (ADT) value for 1985, therefore, the vehicular volume projected in 1965 could not be compared against actual data. It is important to note that even though accurate ADT numbers could not be found, there is no evidence to suggest that the embankment failure was in any way associated with excessive vehicular traffic. The posted speed on this section of highway is 40 M.P.H. and a typical section of the roadway can be found in the ODOT Construction Plans, 1965.

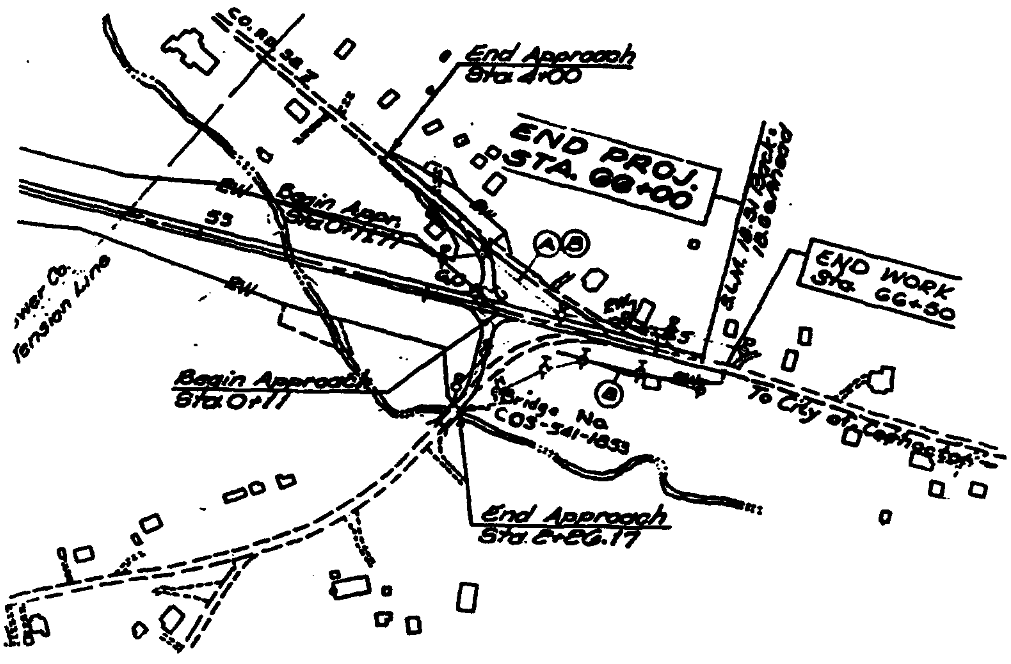

Figure 2.1. SR 541 Site Map Showing Station Locations.

According to Soil Survey of Coshocton County, Ohio (1993) and accompanying detailed soil maps, the soil on which the embankment is constructed is most probably classified as Guernsey silt loam (Gu). A typical profile starting from the subsoil and going to the substratum would be seen as "a friable silty clay loam" that turns to "a light olive brown, mottled, firm, shaly silty clay." (Soil Survey of Coshocton County, Ohio, 1993). Typical soil parameters include moderately low permeability and high shrink-swell potential. The major behavioral concern, as pointed out in the Soil Survey, is the shrink-swell potential and low strength of this soil. The report recommends that an artificial drain be installed if the material is used under roads and streets. According to the Soil Survey (1993), the material of which this embankment is constructed is classified as Udorthents, loamy. This class of soil is often used in construction areas and around factories and highways. Typically, the upper 2 meters of this material is a silty clay loam with a low permeability. The characteristics, as described by the soil report, match closely the material that was excavated in the field.

Table 2.1. SR 541 Design Designation.

Current ADT	710	1965
Design Year ADT	1065	1985
Directional Distribution	Equal	
Percent B&C Trucks	32%	
Design Speed	40 M.P.H.	

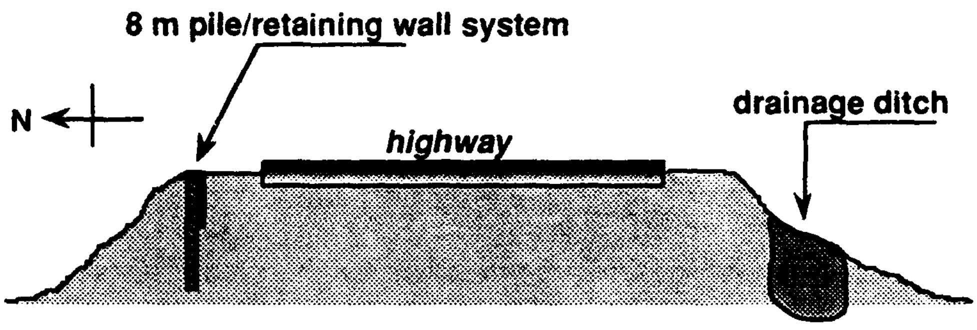

Figure 2.2. SR 541, 1986 ODOT Repair.

There is very little repair history for this section of roadway for the first 20 years after construction was completed. In 1985, road surface settlement was noticed around Station 55+00. The road settlement became progressively worse and was accompanied by some indications of a rotational slide. Reportedly, the base of the embankment was covered with small traverse ridges and there was evidence of soil upheaval. In 1986, an ODOT repair team attempted to correct the problem and prevent any further deterioration by digging a 4 meter deep trench at the top of the slope for a distance of approximately 25 meters and driving a series of 8 meter piles. Next, sections of guardrail were attached to the piles that effectively created

a 25 meter long, 4 meter wide retaining wall. The trench was then backfilled and the highway surface repaired. Another trench was excavated on the south side of the roadway and a drain was installed to intercept surface and subsurface flow associated with the adjacent hillside. Figure 2.2 depicts that repair effort. The repair seemed to work and nothing further was done to this section of highway for the next three years.

In the summer of 1989, realizing that the pile-supported retaining wall had failed to prevent the road surface from settling, a more comprehensive repair was ordered. The eventual uncovering of the piles revealed how dramatic the embankment movement was. The base of the piles, which was approximately 8 meters below grade, had moved approximately 2 meters down slope. This equates to a rotation angle for the piles of almost 20 degrees. It is safe to assume that the tops of the piles remained relatively stable as they were found in their original drive locations. Additionally, the tops of the piles had guardrails attached to them to act as a retaining wall, which greatly reduced any horizontal movement. The piles were removed and the embankment material was excavated to a depth of 13 meters below the road surface. Two drains were installed near the top of the embankment to intercept the water which had been detected during excavation. The excavated material was replaced in controlled lifts and the slope was brought back to its original condition (2:1 slope). These drains are still active, particularly after periods of heavy rainfall or snow melt (Newhart, 1994).

On 17 May 1993, the department of transportation was made aware that the road surface was again exhibiting signs indicative of another rotational slide. As before, at the base of the embankment was a zone of earth flow in which material was rolled over and a small traverse pressure ridge was found. The top of the slope suffered significant settlement, revealing a nearly vertical wall, with pieces of earth gathered next to the scarp. By the end of the summer, the shoulder of the westbound lane had settled up to 1 meter and was progressing towards the center of the road. Concurrently, the Ohio State University was discussing with ODOT the possibility of utilizing dry FGD by-products in state highway construction and repair projects. An agreement was reached to use the Tidd plant PFBC ash to repair the SR 541 slide and the project became the subject of this study (Newhart, 1994).

2.2 Project Report

For simplicity of presentation, the project has been classified into three phases. Phase 1 was the excavation of the embankment material and underlying natural soil. Phase 2 was the placement of the FGD by-product and the replacement of the excavated material and select fill. Phase 3, which is in the very early stage of implementation, is the post construction, long term monitoring of field instrumentation placed in the slope in order to detect any gross movement.

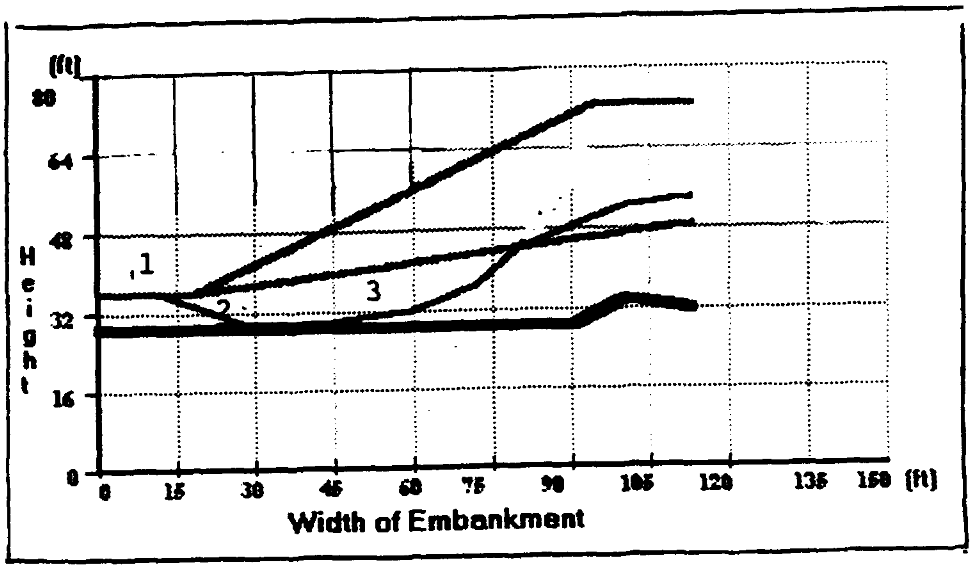

Figure 2.3. SR 541, Typical Section.

2.2.1 Phase 1

By the summer of 1993, the same section of SR 541 was again failing, prompting the Ohio Department of Transportation to place warning barrels around the collapsed shoulder (Figure 2-4). By the end of September, the roadway had been completely closed to through traffic and the repair project planned out. Phase 1 began at the end of September and continued for approximately eight weeks. The excavation operation, as well as all other phases of the project, were conducted solely by personnel from ODOT's District Five Special Projects Branch and the Coshocton County Division. The majority of the equipment used, which included bulldozers, a grader, dump trucks, self-loading scrapers, a roller, and excavators, were state owned. The equipment that the state did not have, or could not readily provide, was locally contracted. It is important to highlight the fact that special equipment was not necessary to place of the clean-coal technology by-product.

Figure 2.4. SR 541, Looking West, Barrels on Collapsed Shoulder.

The repair crew started the project by flattening out the traverse ridges which had formed at the base of the embankment. As they began to excavate, a layer of moist grayish (olive brown) clayey-shale was exposed (location #1 on Figure 2.3). After a few meters of excavation, the grayish soil being removed was completely saturated. The elevation of the saturated layer seemed to coincide with the elevation of the original natural ground surface beneath the embankment as depicted in the original construction plans (location #2 on Figure 2.3). The weight of the vehicles operating on the soil caused it to pump, temporarily halting the operation while a trench was dug to find the source of the excess water. When the trench was approximately 2 meters deep, a spring was discovered (Figure 2.5).

Water rushed out of the new opening for 15 minutes and eventually settled into a steady flow that continued for 3 weeks at a rate of approximately 1.5 - 2.0 l/s (Newhart, 1994).

Figure 2.5. Trench at Base of Embankment, Released Spring.

Realizing that the overall stability of the embankment was a function of both the embankment material and underlying foundation material (NCHRP, 1971), a complete excavation was ordered. The excavation, which had started at the north

edge of the highway, had progressed across the road to its southern edge. The bottom of the excavation was approximately 13 meters below the original road surface (location #3 on Figure 2.3). At that elevation, the soil was a combination of exposed shale and a very stiff blue-gray shale clay. The saturated fill material, which had very poor strength properties, continued to fail as the surrounding material was excavated. This led to the propagation of failure that eventually resulted in an excavation effort much larger than originally planned (Figure 2.6).

Figure 2.6. SR 541 Embankment, Sliding During Excavation.

In addition to having to excavate more material than originally expected and having to contend with an unusually wet season, the long completion time of Phase 1 can be attributed to the crew working only part time. By late November, the total volume of material removed was approximately 9,000 cubic meters. Approximately half of this material was stockpiled on site to be used later in Phase 2. The balance of the soil, primarily saturated clay, was transported off-site.

2.2.2 Phase 2

The second phase of the project began on 23 November 1993 with the placement of the gravel, filter fabric and drain tile used to drain the hillside (Figure 2.7). A drainage pipe was placed along the base of the FGD by-product buttress for its entire length. The drainage pipe outlet was positioned so that it would empty into a stream near a culvert that runs underneath the roadway at Station 54+00 (Figure 2.8). The filter fabric was attached to the exposed face of the embankment in overlapping strips for a total height of approximately 5 meters. The flow, which has been relatively constant since the drain was installed, is approximately 0.5 l/s. It may be hypothesized that the system is operating as intended and the drainage layer is preventing much of the water from reaching the FGD by-product. Water that reaches the retaining layer will likely find an alternate path, as the permeability of this material is as low as 9.1×10^{-10} cm/sec (Bigham et al., 1993).

Once the geotextile was in place, the ODOT Special Projects team began placing the dry FGD by-product, which had been stockpiled on site during the latter stages of Phase 1. Self loading scrapers delivered the material to the excavation as bulldozers spread it evenly over an area 12 meters wide by 30 meters long. The first lift of approximately 60 cm, was placed and rolled at the end of the first day.

Figure 2.7. SR 541 Embankment, Placement of Draining Material.

The next morning, as the scrapers were delivering the FGD by-product for the subsequent lifts, the drivers noticed that the initial lift was so hard that the vehicles were not leaving tire tracks on the surface. In a little over 12 hours, the Tidd ash was strong enough that 40 metric ton scrapers could move freely over it without any noticeable settlement (Newhart, 1994).

Figure 2.8. Drain Outlet.

Having never utilized clean-coal technology by-products in a project, personnel from the Special Projects division of the ODOT were developing their construction techniques as they were placing the by-product. Instructed that the material had to be hydrated to induce strength gains, the crew placed a pump in the stream and added water to the FGD material as it was placed. A roto-tiller was used to mix the top 20 cm of each layer as water was added and the surface was then

compacted by a pad foot roller (Figure 2.9). There were no stringent controls on the depth of the lifts nor on the quantity of water added. The amount of water added in the field (24%) exceeded the optimum water content (18%) as determined in the lab (Bigham et al., 1993). ODOT field personnel adjusted the size of the lifts and the amount of water added to suit the prevailing conditions. The important point to be addressed is the material has a wide workable range and does not have to be mixed with laboratory precision to yield excellent strengths.

Figure 2.9. SR 541 Embankment, Roto-Tiller.

The FGD buttress was built up until the end of November, when approximately 4 to 5 meters of the material had been placed. Atop the FGD, the original

embankment material was replaced throughout December in controlled lifts that ultimately totaled another 3 to 4 meters. See Figure 2.10.

Figure 2.10. SR 541 Embankment, FGD By-Product Layer.

A second source of moisture, at a depth of 6 meters below the original road surface, was detected in the hillside. Additional excavation exposed another potential spring, prompting the installation of two additional drains. The procedure for drain installation was similar to that described earlier. On 3 January 1994, the second FGD by-product layer was placed at this elevation. This layer extended from the center of the road to within 2 meters of the leading edge of the original embankment. This reinforcing layer measured 1 meter in depth, 10 meters

in width, and 20 meters in length. The top of the layer is approximately 5 meters below the original roadway surface.

Figure 2.11. SR 541 Embankment, Borrow Pit.

To raise the embankment from this level up to the original grade, a grayish-brown clayey-shale was brought in from a borrow area two miles west of the construction site (Figure 2.11) and mixed with the original embankment material. The base

course of the repaired area was brought back to original grade in early February, and the FGD by-product was mixed with aggregate to form a temporary wearing course. The road was opened to traffic in early April. To date, the slope has not been seeded and a permanent wearing course must still be placed (Figure 2.12).

Figure 2.12. SR 541 Embankment with Temporary Wearing Course.

2.2.3 Phase 3

During the excavation and reconstruction of the embankment, there has been regular monitoring of the water quality, both upstream and downstream of the project location. These measurements have consisted primarily of pH, but in addition, several water samples taken from the drain lines under the FGD material and from the adjacent stream have been analyzed for the total dissolved solids

(TDS), total alkalines, hydroxide alkalines, SO_4^{2-} and Cl^-. The same tests were conducted on water samples taken after the FGD was placed and the embankment brought back to its original condition. The data can be viewed in Table 2.2.

Table 2.2. SR 541, Pre and Post Construction Water Quality Values.

	pH	TDS	Total Alk as $CaCo_3$	Hyr. Alk. as $CaCo_3$	SO_4^{2-}	Cl^-	Hardness as $CaCo_3$
Pre-Const.							
Location #3	8.3	468	13	0	312	245	N.A.
Location #7	7.7	414	11	0	40	270	N.A.
Post-Const.							
Location #3	6.3	578	400	0	1100	26	875
Location #5	7.0	296	840	0	820	20	875
Location #7	6.6	226	600	0	1050	20.5	1125

Note: All values expressed in mg/l.

It can be seen that there are not great variances in pH nor TDS, which seem to be within an acceptable range of fluctuation associated with the stream and location of measurements. However, there is a significant rise in total alkaline measured in mg/l as $CaCo_3$. Prior to the placement of the FGD material, this value averaged 12 mg/l, compared to an after FGD material placement average of 450 mg/l. Similarly, SO_4^{2-} increased from an average of 176 mg/l to an average of 1294 mg/l. Conversely, the amount of Cl^- dropped from an average of 258 mg/l to an average of approximately 34 mg/l. Because the volume of stream flow is so much greater than the volume of water being expelled through the drain, the total system should be unaffected by the increase in measured $CaCo_3$ and SO_4^{2-}. However,

the increases in these values support the need for long-term water quality monitoring.

Field pH values were monitored on a regular basis from the pre-construction period to the present. These values were collected to determine whether there was a noticeable trend in stream pH values both before and after FGD placement. The values can be viewed in Table 2.3.

Table 2.3. SR 541, pH Values.

Location	Nov 23	Dec 1	Dec 10	Dec 17	Dec 22	Jan 27	Feb 18	Mar 8	Mar 15	Average	STDEV
1	7.3	8.0	8.4	8.3	8.3	7.1	7.4	7.5	7.0	7.7	0.55
2	7.1	8.1	8.4	8.3	8.1	7.3	7.1	7.3	7.0	7.6	0.58
3	7.0	7.2	8.0	7.9	8.0	7.2	7.1	7.3	7.0	7.4	0.43
4	6.7	6.9	7.3	7.4	7.6	7.2	7.1	7.3	7.0	7.2	0.27
5	6.7	6.8	6.9	6.9	7.0	6.8	6.9	7.1	6.8	6.9	0.12
6	7.0	7.1	7.2	7.3	7.2	7.2	7.1	7.3	7.0	7.2	0.11
7	NR	NR	NR	NR	NR	7.2	7.3	7.3	7.1	7.2	0.10
8	NR	NR	NR	NR	NR	7.2	7.4	7.4	7.2	7.3	0.12
Average	7.0	7.4	7.7	7.7	7.7	7.2	7.2	7.3	7.0		
STDEV	0.23	0.56	0.65	0.57	0.52	0.15	0.18	0.11	0.11		

Eight locations were identified and used as reading points. Three of the locations were significantly upstream of the lower drain outlet and were chosen because they should not be affected by any FGD by-product related drainage. These locations may be viewed as points #1, #2, and #3 on Figure 2.13. It is also highly unlikely that these locations are in anyway affected by FGD material influenced runoff, since the top of the FGD buttress is several feet below the slope surface and only a few feet above the stream level. One location was selected in the outlet drain and

directly measured the pH of the water being expelled from the drainage material installed at the base of the lower buttress and can be viewed as point #5 on Figure 2.13. Measurement points were established on either side of the drain, approximately 1 meter upstream and downstream and are points #4 and #6 on Figure 2.13, respectively. Presumably, the eddies caused by the rocks within the stream and the natural terrain of the area caused this region to act as a mixing area. Another measuring point was selected approximately 50 meters downstream to examine the effects of the drainage water on the stream system as a whole, and is seen as point #7 on Figure 2.13. It can be inferred that the water from the drain and the natural stream water were well mixed by this point in the system. Finally, point #8 (Figure 2.13) was selected in an adjacent stream prior to its confluence with the stream of interest. This was done to measure the pH of a stream that was in no way related to the FGD by-product demonstration project and to establish a normal pH fluctuation range. Figure 2.14 shows the stream and one of the reading locations.

Two pH meters manufactured by Omega were used to take all of the measurements. The PHH-3X is capable of measuring water temperature as well as pH, whereas the PHH-1X only reads pH values. The issue of temperature became important once it was noticed that both pH meters were somewhat sensitive to the temperature of their surroundings. Table 2.4 shows what each meter read in a certified 7.0 pH solution at varying temperatures. The values in Table 2.3 have been adjusted to reflect corrections for temperature to the pH of the buffer itself.

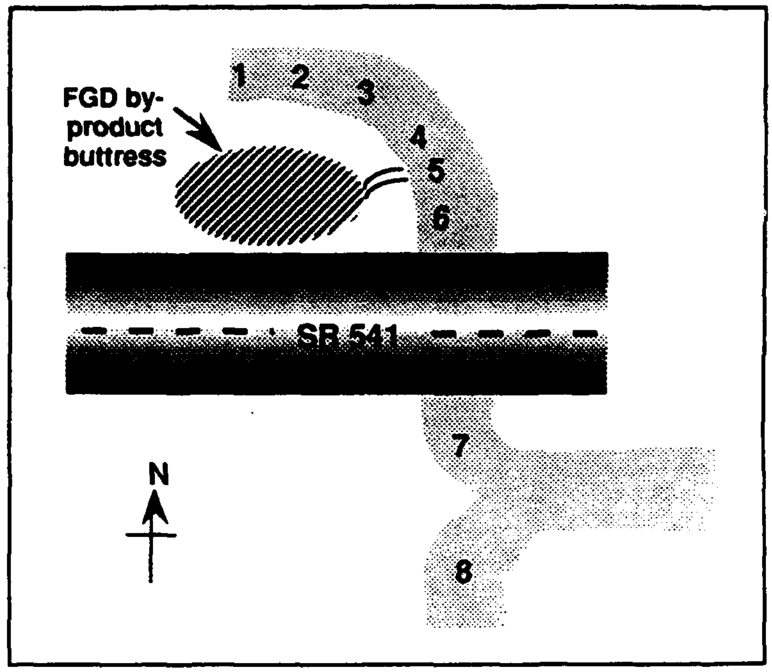

Figure 2.13. SR 541 Embankment, pH Reading Locations Along Stream.

No significant differences have been observed in the pre-construction and post-construction values and the differences that have been noted fall within an accepted range of normal stream fluctuations. The stream that runs along the construction area has not experienced fluctuations greater than those measured in the adjacent stream. This is encouraging, as large fluctuations in stream pH would be detrimental towards the future use of this type of material in water infiltrated areas. On average, the pH of the water directly exiting the drain is slightly lower than the stream mean (6.9 v. 7.4).

The fact that the average stream pH is above 7.0 suggests that there may be some type of limestone outcrop that the water flows over, or that there may be another

source that consistently raises the overall stream pH. The relatively consistent drain outlet readings (standard deviation = 0.12) suggest that the underground spring that infiltrates the embankment is not affected by the same source that increases the pH in the surface stream.

Photograph 2.14. pH Reading.

Further, it may be surmised that the spring water is exiting the drainage system prior to contacting the FGD material, since contact with the lime enhanced by-product would increase pH levels. Considering these normal pH values and the volume of water estimated to be flowing from the outlet, it seems the drain is performing up to expectations. With respect to pH values, and from all other available data, it may be inferred that the clean-coal technology by-product used in

the reinforcement of this embankment does not have any negative environment effects.

Table 2.4. pH Value Corrections.

Temperature	**PHH-3X**	**PHH-1X**
(°F)	(pH)	(pH)
62	7.1	7.1
51	7.2	7.0
47	7.3	7.0
42	7.4	7.0
39	7.5	6.9
36	7.5	6.9
33	7.6	6.9

The instrumentation of the slope and an analysis of the measurements made is expected to continue over the next three years. Additionally, water measurements to study pH, total dissolved solids (TDS), total alkalines, hydroxide alkalines, and SO_4^{2-} and Cl^- will be taken. The proposed instrumentation of this site would incorporate inclinometers, piezometers, and various deformation gauges. This is further discussed in Chapter 4, Long-term Monitoring.

CHAPTER III.

SR 541 EMBANKMENT: STABILITY ANALYSIS

3.1 Introduction

The stability of the embankment, both before and after reconstruction, is investigated in this chapter. FGD material properties used as input in these analyses were obtained from earlier laboratory studies conducted in the Civil Engineering Department. A complete record of these tests is presented in Bigham et al., 1993. Soil parameters for the embankment material and the natural earth were established from field tests and from fundamental laboratory analyses. The strength values for the crushed shale and clay mixture used in the reconstruction of the embankment are unknown and are assigned the more conservative values of the embankment material. A table of soil properties is presented in Section 3.3.2.

There is no greater prediction of soil strengths than full scale testing. Though the collapse of the SR 541 embankment was an unfortunate event, it afforded the author the opportunity to back-calculate probable strength values for the soils in the failed region. The estimated soil strength values were calculated using a

stability program with the existing slope geometry and details of the failure plane as input. The predicted strength values were combined with field and laboratory determined parameters, including unit weight and plasticity index, and compared against published values for similar material.

Though this method does not yield exact soil strengths, it does allow one to predict a range of possible soil parameters and adjust them to reproduce the actual outcome. A shortcoming in this method is that in non-homogeneous systems, strength parameters can be decreased in one soil type and raised in another and still yield the same factor of safety for the system. However, drastic strength increases or reductions for soils that deviate from published values for similar materials immediately becomes apparent. This preserves the integrity of the method and yields respectable values for the system as a whole. For this demonstration project, the non-homogeneity of the embankment material and the lack of access to intact samples supports the conclusion that more accurate soil strength values were not attainable from any other method. The importance of this research is not to determine the precise material properties of the embankment and underlying earth, but to analyze the suitability of FGD by-products as an embankment stabilizer. The issue is addressed in this chapter.

The factors that lead to the failure of the SR 541 embankment can be classified as either those causing increased stress or those causing a reduction in strength. The factors that cause increased stress include increased unit weight of soil due to saturation, increased external loads and shock loads, and the steepening of slopes

(Schuster and Krizek, 1978). Other than increased unit weight of the saturated soil, it is not probable that any of these other events were significant factors in the embankment failure. It does seem likely that the failure was due to the loss of strength of the fill material and the underlying natural soil. Soil strength losses can be attributed to the absorption of water, increased pore pressures, cyclic loading, freeze-thaw action, loss of cementing material, the weathering process, and st th loss associated with excessive strains (Winterkorn and Fang, 1975). Th gh many of these agents may have acted in concert, it seems most probable that water was the main cause of failure in this embankment.

3.2 Manual Analysis

To support this hypothesis, a slope stability analysis was required. The most common methods of slope-stability analysis are based on limit equilibrium. In this type of analysis, the factor of safety is approximated with respect to the slope's stability by examining the condition of equilibrium. An incipient failure is postulated along a pre-defined failure plane and the strength that is necessary to maintain equilibrium is compared to the available strength of the soil. All limit equilibrium problems are statically indeterminate and involve the judicial use of simplifying assumptions, since the stress-strain relationship along the assumed failure surface is not known (Bishop and Bjerrum, 1960).

When using the limit equilibrium method for stability analysis, either effective or total stress may be used. If the effective stress method is used, pore pressure along

an assumed failure surface are estimated for use in the analysis. Shear strength becomes a function of the effective strength parameters. In the laboratory, effective stress parameters are obtained from either consolidated drained shear tests or from consolidated undrained shear tests with pore pressure measurements. In the total stress method, the shear strength is given in terms of total stress and the laboratory tests are intended to simulate the actual field conditions of the embankment (Bishop and Bjerrum, 1960).

Of the many limit equilibrium methods available, Bishop's Simplified Method of Slices was chosen for its completeness and ease of use. Bishop found that by including horizontal side forces to compute the normal force (Pn) and also satisfying the overall moment equilibrium, the resulting factor of safety was only slightly less than values calculated in more rigorous methods of Morgenstern and Price(1965) or Spencer (1967). The failure arc predicted by Bishop's Simplified Method of Slices has been found to compare well with actual failure surfaces. Also, Bishop's Simplified Method was chosen because a knowledge based system designed to analyze slope stability using this method was utilized under this research. The knowledge based system used to analyze this demonstration project is described in Section 3.3, Computer Aided Analysis.

Prior to undertaking the more sophisticated automated slope stability study, a manual analysis was conducted on the embankment. The configuration of the slope was taken to be identical to the cross section view depicted as Station 55+00 (ODOT Construction Plans, 1965). Using Bishop's method for total stress

analysis, as outlined by Dunn, et al. (1980), a factor of safety of 0.8 was established. Since the embankment was considered undrained, the friction angle was taken as zero and a cohesion value of 19 kPa was used. This method of analysis satisfies only the overall moment equilibrium, neglecting the moment equilibrium for the individual slices. Further, this technique only approximates the force equilibrium of the individual slices. Figure 3.1 depicts a typical slice that includes side forces E and X that represent the horizontal and vertical forces, respectively.

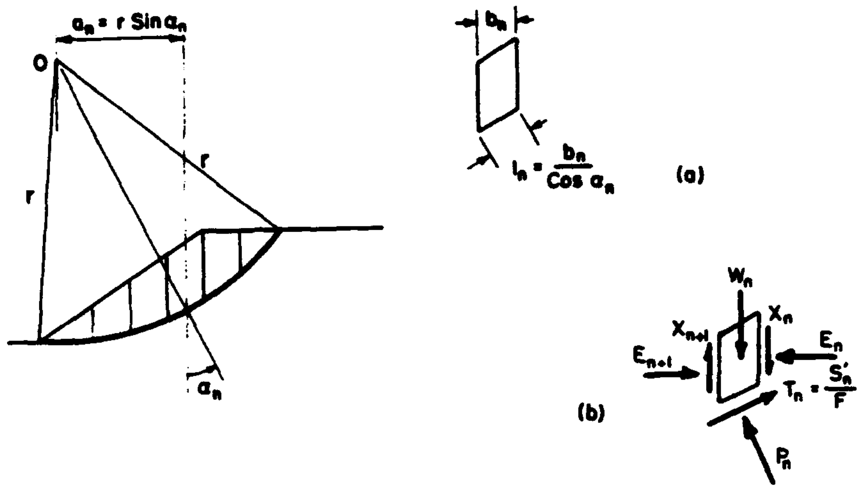

Figure 3.1. Typical Slice.

Each slice is assumed to have the same factor of safety, F, and the same required strength, Tn. Simply stated, the required strength is equal to the available strength of each slice divided by the factor of safety. Bishop simplified the system by canceling the vertical forces [Σ (x_{n+1} + x_n) = 0], and after summing all

appropriate remaining forces in the vertical direction and accounting for the laws of equilibrium, derived the following equation:

$$F = \Sigma \{[c_n b_n + (W_n - u_n b_n)(\tan \Phi_n)] \times \sec \alpha_n / [1 + (\tan\Phi \tan\alpha_n / F)]\} \qquad [3.1]$$

For a saturated soil system, which was a condition that existed in the embankment, the friction angle is assumed to be equal to zero and the factor of safety can be solved for directly. Equation 3.1 reduces to the following:

$$F = \Sigma [c_n b_n + (P_n - u_n b_n)(\tan \Phi_n)] / \Sigma W_n \sin \alpha_n \qquad [3.2]$$

where

c_n = cohesion	F = factor of safety
b_n = width of slice	P_n = normal force
W_n = weight of slice	l_n = length of slice

u_n = average pore pressure at bottom of slice

Φ_n = effective friction angle

α_n = angle measurement of slice position relative to arc center

Letting the friction angle equal zero in Equation 3.2, the factor of safety is merely a function of cohesion, slice width and weight, and the relative position to the slice to the arc center.

Using the ODOT construction plans to establish the slope geometry and field data to position the slide near its original location, an analysis was conducted using the

above procedure. Soil parameters were established from field and laboratory tests and back-calculations, or were taken as conservative values from published sources when they were unknown. It is important to note the exact soil values were not of paramount importance as this computation was made simply to establish whether the occurrence of the slide was supported by data or whether there were some abnormal factors leading to its failure. The calculations indeed indicate that given the slope geometry, soil parameters, and observed slip plane, the failure was not only possible, imminent. For simplicity, a spreadsheet was substituted for the hand calculations and can be viewed as Appendix A.

3.3 Computer Aided Analysis

Due to the increased complexity of the geometric configuration for the FGD by-product reinforced embankment, a more sophisticated stability analysis had to be conducted. Because of its acceptance in industry as well as its attractiveness as a teaching tool, PC STABL (Lovell, 1988) was chosen as the slope stability analysis program. Further, this research program was being conducted simultaneously with the development of a knowledge based system, that included PC STABL, in an overall highway design program.

The STABL computer program was written in FORTRAN IV for general solutions of slope stability problems by utilizing a two-dimensional limit equilibrium method. The program offers the user the choice of several popular analytical approaches (the Simplified Bishop Method, the Simplified Janbu

Method, and the Spencer Method) and also allows the operator to specify a particular failure surface. After all of the necessary information is input, the program generates a series of potential failure surfaces, each with a computed factor of safety. Realistic field conditions can usually be specified since the program allows for heterogeneous soils systems, anisotropic soil strength properties, excess pore water pressure, and static ground water tables.

3.3.1 Data Preparation

The profile of the embankment has to be plotted on a grid, with coordinates marking surface and subsurface incongruencies. Once the geometry is established, all of the different soils that make up the embankment have to be identified and appropriate parameters have to be assigned. The program requires the location of the water table has to be defined and the regions of possible failure origination and termination need to be established. Additionally, external loads can be specified, which have not necessary in any of the following scenarios.

PC STABL was used in the decision support system developed by Kim (1994). This program incorporates expert experiences, heuristic judgment, and calculated results from analytical programs (Kim, 1994). The objective of Kim's Intelligent Decision Support System for Highway Embankment Design (IDSSHED) is to assist in the design of FGD by-product enhanced embankments by evaluating the chosen FGD by-product and by performing the design calculations. The system also provides technical information necessary to incorporate FGD materials into

highway slope stability as a function of factor of safety and provides settlement calculations.

The system allows user specified failure surfaces and permits true field conditions to be represented. The program asks for information such as soil parameters, slope geometry and profile, and water table location, which are gathered and compiled in input files. Details of file preparation can be found in the PC STABL 5M User's Manual (1988) or Kim (1994). After all of the necessary information is input, the program generates a graphical representation of the embankment and the possible failure surfaces, each with a calculated factor of safety.

3.3.2 Multiple Scenario Analysis

Several potential failure scenarios were developed to analyze the effects of the FGD by-product on the overall stability of the embankment. The input files for each of these scenarios can be found in Appendix B. Each input file contains the points that define the embankment geometry, soil strength parameters and unit weight, and the water table locations. Included as the soil parameters are the saturated unit weights and the strength parameters, cohesion and friction angle. The soil parameters used in the analysis were an average of values obtained from field measurements and from laboratory tests. The embankment geometry and the bedrock location were kept constant in all scenarios. The variables in the different scenarios were the water table location, the soil strength parameters, and the presence of the FGD by-product buttress system.

The soil strength values obtained in the field and in the laboratory were relatively consistent, with standard deviations all within reasonable limits of engineering accuracy. Table 3.1 shows a summary of the different input values and the standard deviations associated with them. The parameters used in the various analyses were all within established ranges for the types of materials in the embankment.

Table 3.1. SR 541, Soil Properties

SOIL	**C** (kPa)	**Φ** Deg.	**γ_S** (g/cm^3)	**LL** (%)	**PI** (%)	**Moisture** (%)
Embankment Mat.	38	20	1.91	34	14	17
Natural Material	19	0	1.91	Unk.	Unk.	17
FGD Material	4500	0 - 35	1.76	Unk.	Unk.	24
Borrow	0 - 38	0 - 35	1.91	Unk.	Unk.	Unk.

3.3.2.1 Original Embankment

Figure 3.2 depicts the original embankment and the most critical failure surface. It shows a rotational failure starting approximately 2 meters from the edge of the highway and terminating near the toe of the slope. The failure surface is a circular arc that passes through the embankment material and natural soil, tangent to the underlying bedrock. The calculated factor of safety associated with this slope is approximately 0.95.

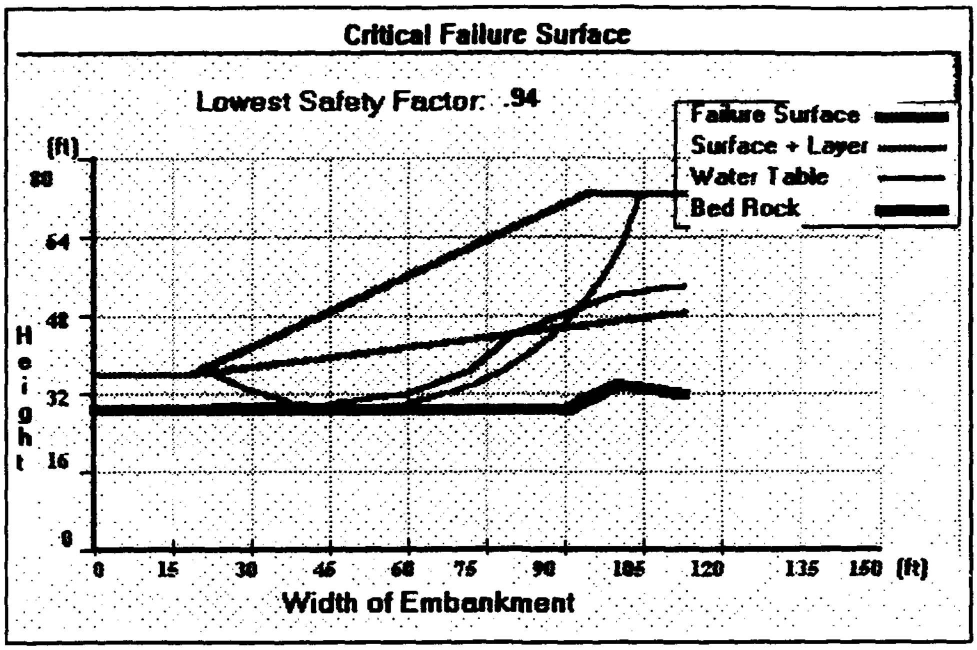

Figure 3.2. SR 541 Embankment, Original Profile, Critical Slip Plane.

Figure 3.3 is a representation of what the actual failure surface looked like. The shape of the failure arc is very similar to the one predicted by the program, differing only in its coincidence with the water table and slip initiation and termination points. It is possible that the failure surface generated by the program and depicted as Figure 3.2 is what actually occurred in May, 1993. What was observed and measured later that year is what is depicted by Figure 3.3. The geometry of the failure plane in Figure 3.3 could simply be a subsequent surface of the failure plane depicted in Figure 3.2. The four months between the first recognized failure and the start of the corrective action could account for the slightly different failure initiation and termination points, as well as the position of

the failure plane and the elevation of the water table. Given the variable nature of soil and the possible alternate locations for the water table, the author feels the results match well. It may be inferred that the two failure surfaces are nearly identical, as is the difference in the calculated factors of safety (0.95 v. 1.1).

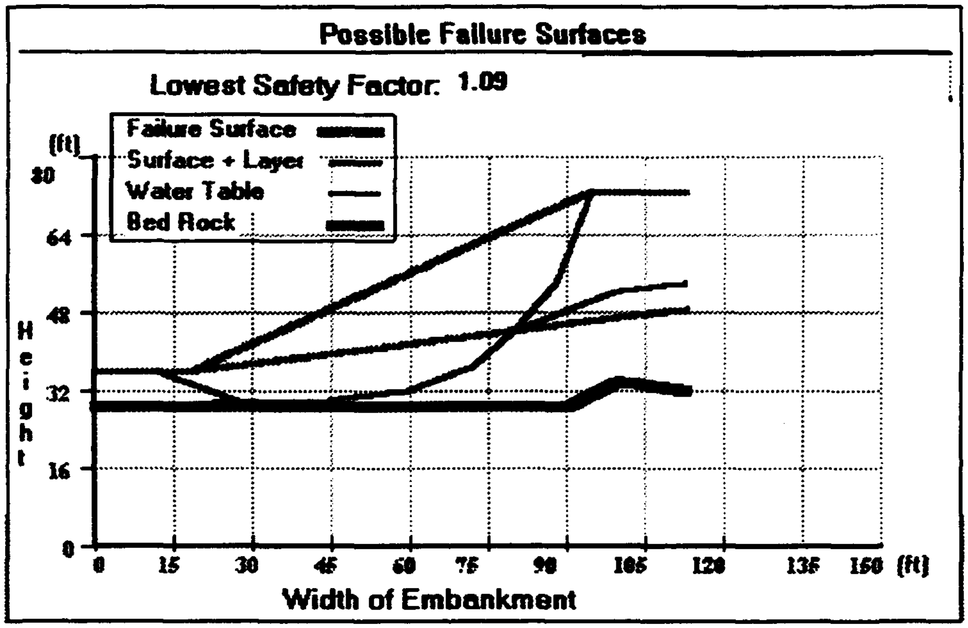

Figure 3.3. SR 541 Embankment, Original Profile, Actual Slip Plane.

It is possible that the slope did not exhibit any indications of failure prior to 1985 because the fill material was not retaining the water that was later found to be a trapped aquifer. As the manual method used in Section 3.2 suggested, if field conditions were considered drained, then the corresponding factor of safety would

be close to 1.8. By reducing the friction angle to zero, the factor of safety dropped in the manual method to 0.8 and in the STABL calculations to 0.95.

3.3.2.2 Original Embankment, Drain Installed

The next scenario involved modifying the existing embankment geometry by adding the effects of an artificial drain. The intent of this exercise was to determine whether the FGD by-product was needed, or whether traditional drain installation would solve the slope stability problem. Figure 3.4 shows the critical failure surface with the input file simulating a drain between the natural soil and the underlying bedrock. There appears to be no difference in the predicted critical failure surface between this scenario and the original condition.

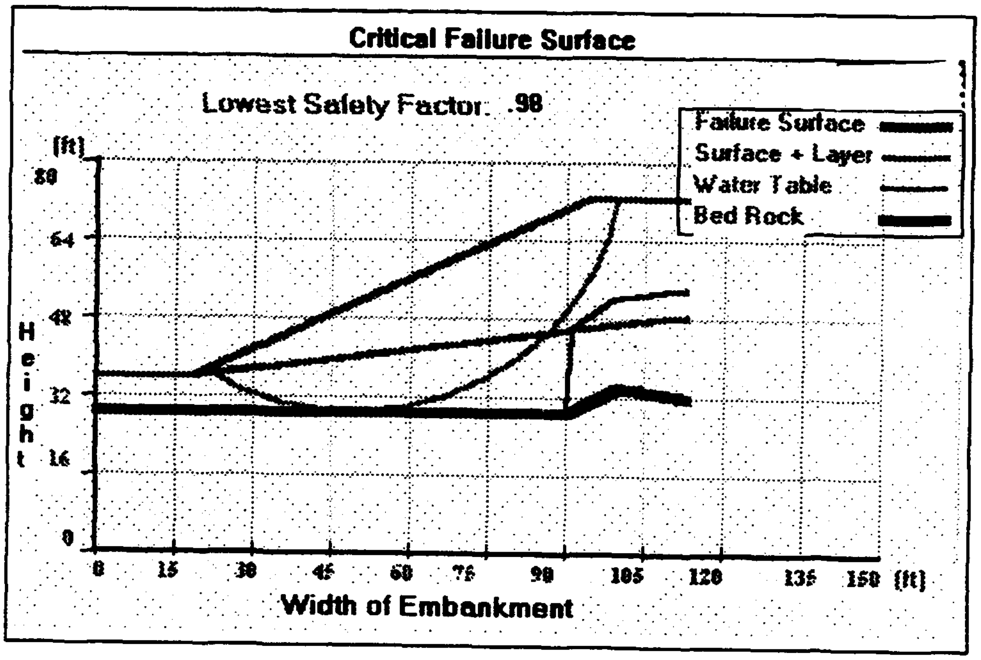

Figure 3.4. SR 541 Embankment, Original Profile, Simulated Drain.

The significant information to be gained from this scenario is that removing the material, installing a drain, and replacing the original material to form the identical embankment would do very little for the long-term stability of the slope. If the friction angle of the natural soil is left as zero, then all of the soil's strength must be due to cohesion, which is unaffected by the lowering of the water table. The calculated factor of safety for this scenario, 0.98, is virtually the same as the factor of safety calculated for the previous scenario, 0.95. The slight difference in the values may be attributable to the program generating incrementally different failure surfaces due to the reconfiguration of the input file. It should be noted that the excavation and reconstruction labor costs associated with stabilizing this embankment are relatively independent of which material is chosen for the reconstruction effort.

3.3.2.3 Original Embankment, Drain Installed, Select Borrow.

In the next scenario studied, the existing soil was excavated and replaced with a select borrow material, and a drain similar to the one outlined in Section 3.3.2.2 was installed. For a fill of the same unit weight and cohesion, but having a friction angle of 20 degrees, a calculated factor of safety of two was obtained. It can be shown that while the select fill doubles the factor of safety for this embankment, the same circular failure surface ultimately develops. See Figure 3.5. Though the calculated factor of safety would be sufficient by most highway design standards, it involves the replacement of approximately 8,000 cubic meters of material, which would have a high cost associated with it. Though a drain is installed in this study,

it is possible that the clay-based select material could become saturated and degrade in strength. The calculated factor of safety would continue to approach one as the strength parameters of the borrow decreased, eventually falling below one and failing. This may be a logical explanation for what actually occurred and why the embankment lasted for approximately 20 years without serious incident.

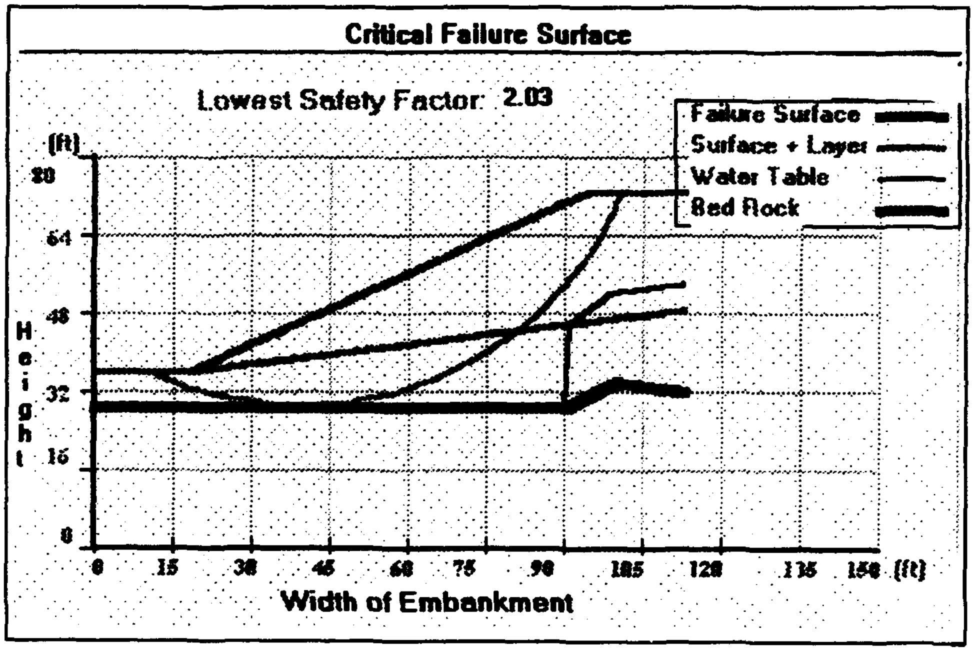

Figure 3.5. SR 541 Embankment, Original Profile, Drained Condition.

3.3.2.4 FGD By-Product Reinforced Embankment

The next step in the stability analysis of this embankment was to incorporate the clean coal technology by-product layers into the input file. The water table in this calculation was left at its original elevation at the center of the embankment and

was quickly lowered once it made contact with the drainage material adjacent to the lower FGD buttress. What water is not intercepted by the filter is assumed to go around the buttress and exit at either the eastern or western edges of the slope. The program is limited to two-dimensional analysis, therefore the water table was simply lowered to the bottom edge of the buttress and maintained at that elevation. Figure 3.6 depicts the profile of the embankment with the two FGD by-product layers included. The larger buttress is primarily in the natural earth material and sits atop the bedrock. The smaller layer, which is approximately 1 meter thick, lies 6 meters below the surface of the highway.

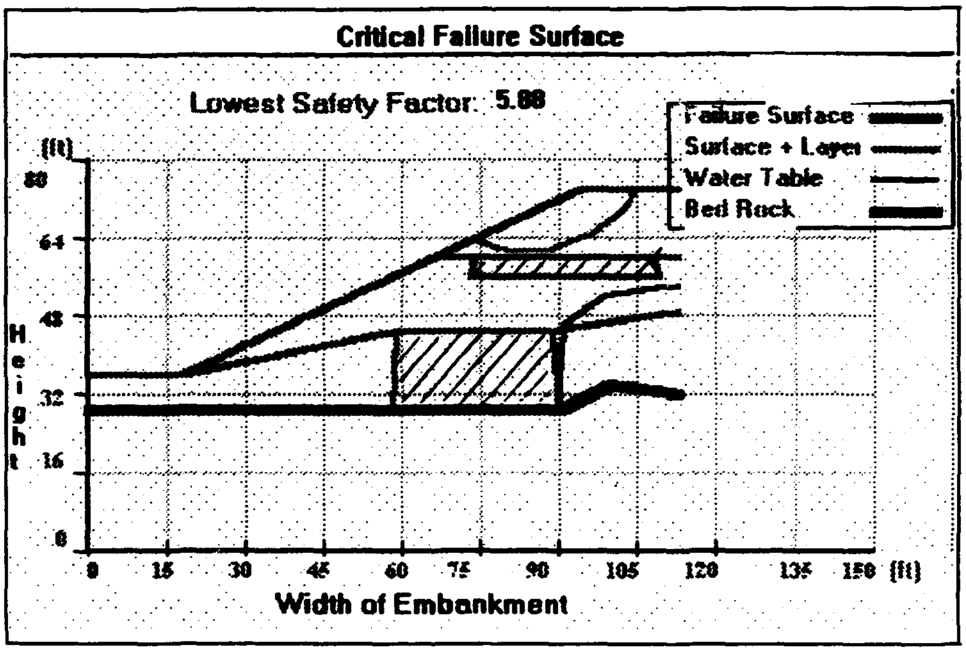

Figure 3.6. SR 541 Embankment, FGD By-Product Reinforcement, Critical Failure.

The friction angle for the natural material was left as zero and the embankment material was assigned its original material properties. The layer atop the second buttress was a combination of crushed shale and select borrow and was given material properties similar to the embankment material. The assigned values proved to be sufficient, as numerous trials showed the shale layer had little effect on the overall embankment stability. The effect of the FGD by-product reinforcement system can best be viewed as Figure 3.7.

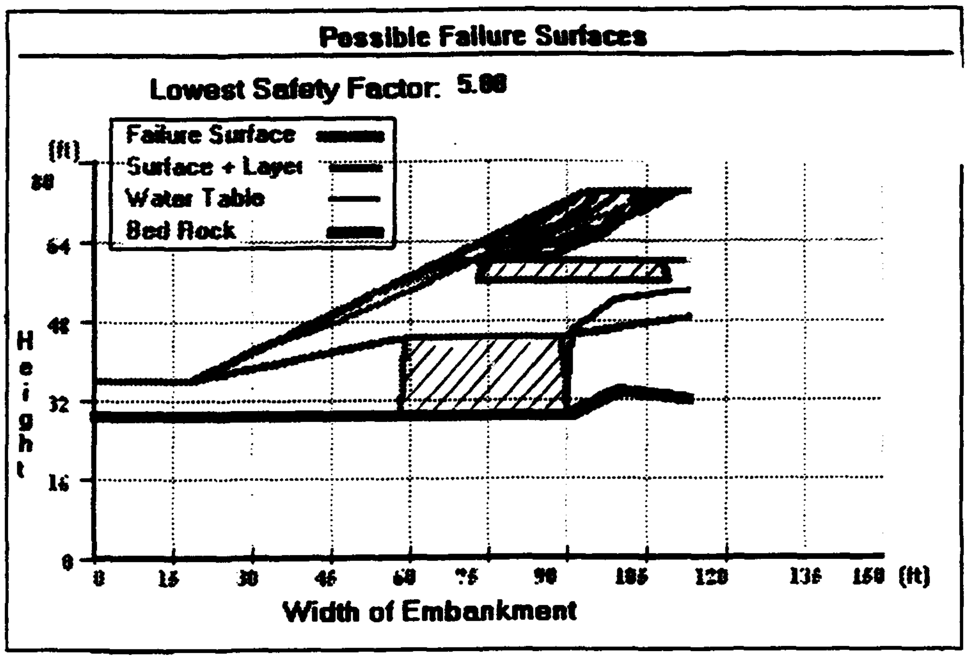

Figure 3.7. SR 541 Embankment, FGD By-Product Reinforcement, All Failure Planes.

The combination of slope stabilizers has forced all likely failure planes out of the FGD by-product and to the surface of the embankment. With the lowest calculated factor of safety of nearly 6, the FGD by-product buttress system has

increased the calculated factor of safety almost three-fold over the select fill scenario. Additionally, less natural earth material had to be excavated as less select material (approximately 2,000 m^3) was used in the re-constructive effort.

3.3.2.5 FGD By-Product Reinforced Embankment, Non-Functioning Drain

As mentioned earlier in this chapter, water can both reduce the strength of a soil by increasing the pore pressure and increase the stress by increasing the unit weight (Winterkorn and Fang, 1975). Since the embankment can be considered a cohesive soil system, excess water can only be viewed as a detriment and an agent that acts to reduce stability. To fully account for the potentially disastrous effects of the water, another input file was created to simulate a failed drainage system. Under this scenario, the lower drain fails and the water table rises to a point that completely encompasses the lower FGD buttress. In reality, the buttress is of finite dimensions and would facilitate alternative drainage paths around its edges, yet this remote event is presented anyway to demonstrate the stability of the reinforced embankment. The water does not infiltrate the buttress, as its permeability is several orders of magnitude lower than the surrounding material. This permits the FGD by-product strength parameters to remain unchanged.

This "what if" scenario is presented as Figure 3.8. The most critical failure surfaces are essentially unchanged, as is the critical factor of safety. All of the possible failure surfaces are still shallow and may be classified as surface slips, not moderate to deep rotational slides. This is not surprising, as all of the potential

failure surfaces shown in Figure 3.7 were well above the water table and the inoperative drain should have had no effect. As expected, the overall strength of the FGD material and the relative positioning of the layers greatly reduce the likelihood that this embankment will fail, even under high water table conditions.

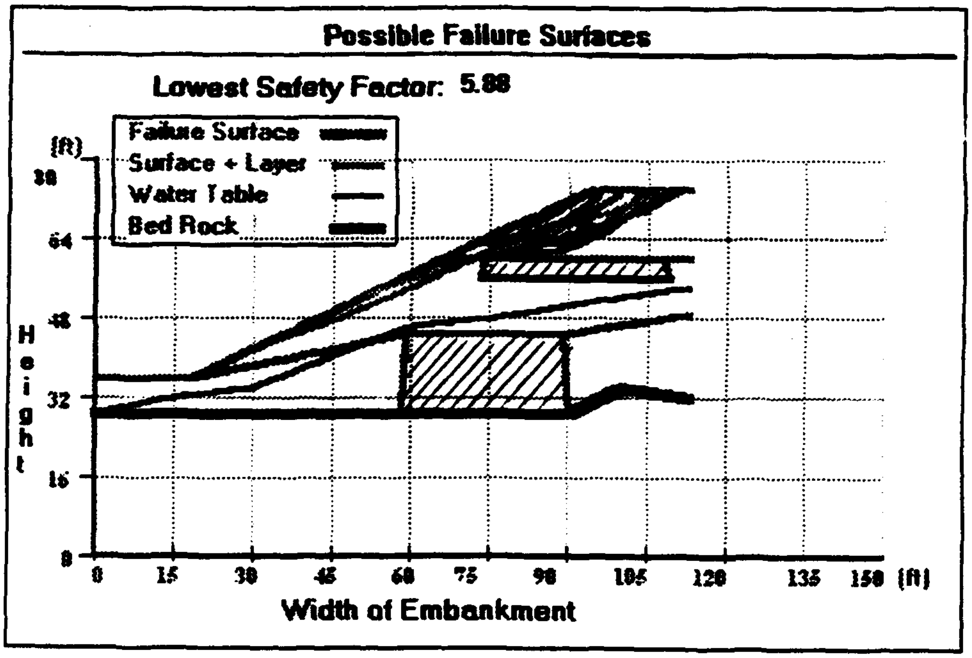

Figure 3.8. SR 541 Embankment, FGD By-Product Reinforcement, Non-Functioning Drain.

3.3.2.6 FGD By-Product Reinforced Embankment, Upper Layer Analysis

To fully understand the effects of the FGD by-product on the entire embankment, an analysis was conducted using the reinforced embankment configuration, modified by replacing the top 5 meters with materials of varying properties. In the first analysis, a crushed shale was used atop the FGD by-product buttress.

Assigning a friction angle of thirty-five degrees ($\Phi = 35°$) and giving the material no cohesive value, the calculated factor of safety for the embankment drops to 1.56. The predicted failure surface, depicted as Figure 3.9, is extremely shallow and can be labeled a surface slide. Conversely, if the material at the top of the embankment is given the properties of a moderately stiff clay in an undrained state ($c = 38$ kPa, $\Phi = 0°$), then the calculated factor of safety climbs to 4.5. Figure 3.10 depicts this scenario and shows how the predicted failure surfaces are slightly deeper than the ones predicted for the cohesionless material. It may be argued that the shaly-clay that was used atop the thin FGD material layer will soon weather to a clay that would have properties similar to those used to generate Figure 3.10.

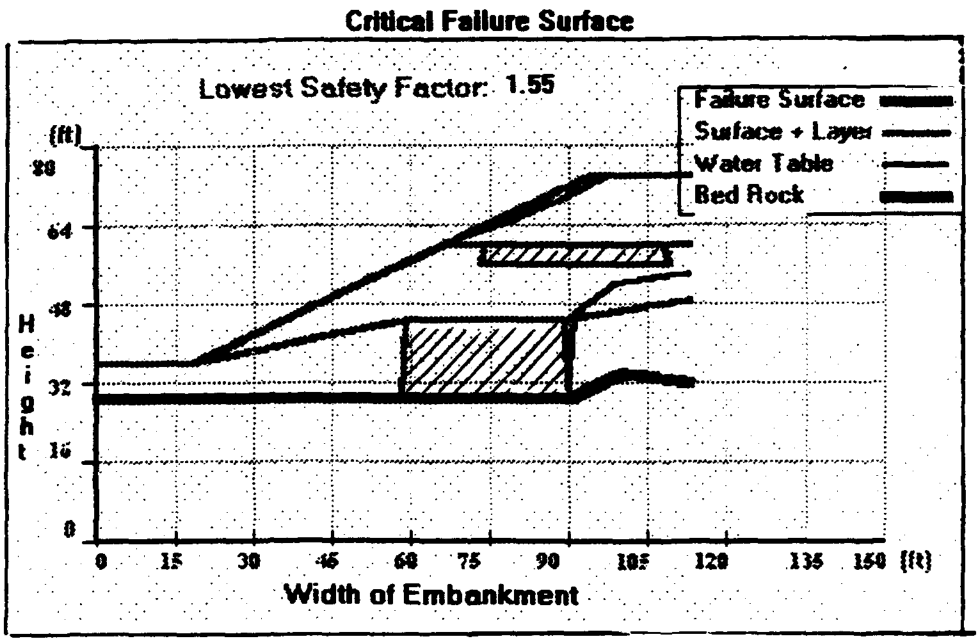

Figure 3.9. SR 541 Embankment, FGD By-Product Reinforcement, Cohesionless Upper Layer.

It may further be speculated that the material used to bring the embankment to grade actually had a cohesive value and a friction angle. Since the borrow, which appeared to be a moderate to stiff clay in its intact form, was mixed with the original silty-clay, the composite material could likely have measurable values for both strength parameters. Such a scenario would produce a failure surface and calculated factor of safety similar to the one predicted in Section 3.3.2.4. Exact values were not assigned to this scenario, as the final calculated factor of safety for the entire embankment would be a function of the assumed strength values for the top layer, and not a function of the FGD material in study.

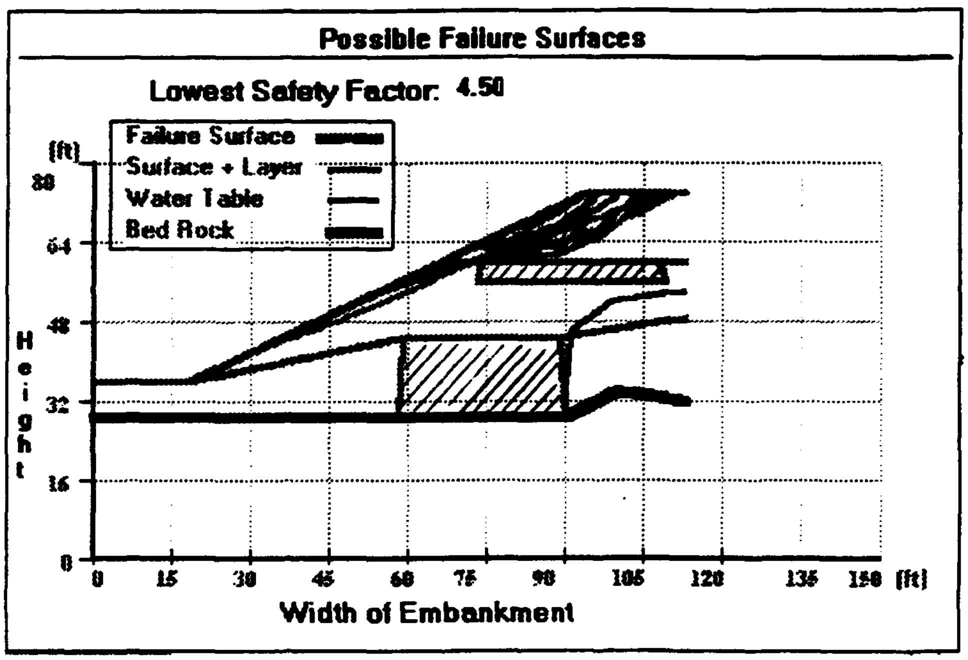

Figure 3.10. SR 541 Embankment, FGD By-Product Reinforcement, Cohesive Upper Layer.

3.3.2.7 FGD By-Product Reinforced Embankment, Specified Failure Plane

To further establish the FGD by-products performance as a slope stabilizer, another input file was created that forced the original failure plane to propagate through the modified embankment profile. Using the embankment geometry from 3.3.2.4, a failure surface similar to the one that was observed (Figure 3.3) was specified. This forced the plane to pass through both FGD by-product layers, as seen in Figure 3.11. As expected, the calculated factor of safety for this scenario is extremely high (58) and reinforces the hypothesis that the observed failure geometry will not reoccur.

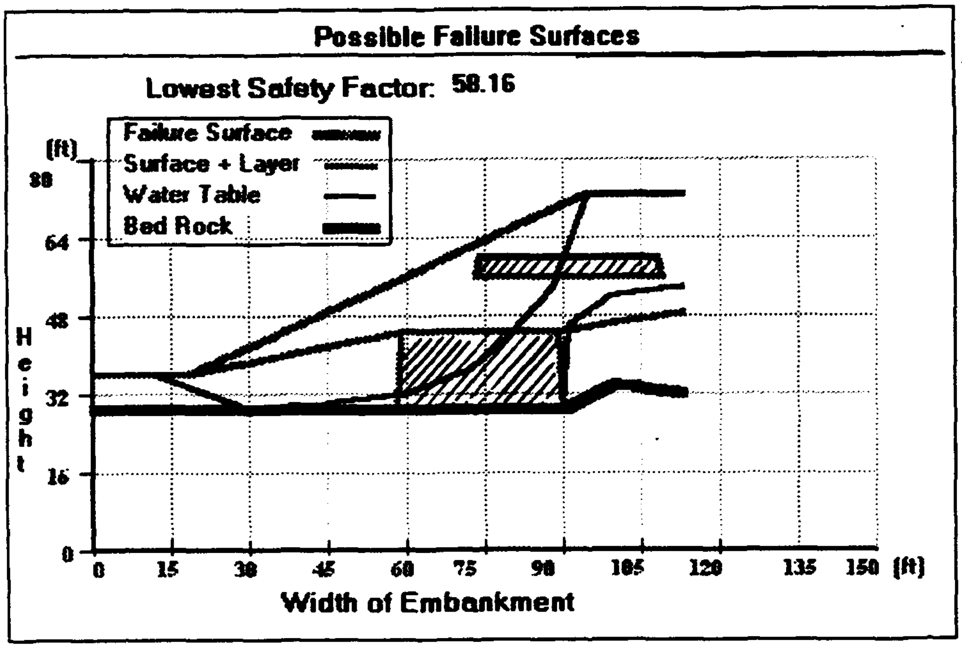

Figure 3.11. SR 541 Embankment, FGD By-Product Reinforcement, Specific Failure Plane.

3.3.2.8 FGD By-Product Reinforced Embankment, Full Depth

As a final scenario, all of the fill material was given the soil properties of the FGD by-product. Though this may not be prudent, due to drainage problems and transportation costs, it was done to study the potential stability of such an embankment. As can be seen in Figure 3.12, the failure surface returns to the position and geometry that we first saw with the original embankment. The difference is in the enormous calculated factor of safety (54). Though this calculated factor of safety is very high, it is essentially the same as the value calculated for the specified failure plane in the previous sub-section. Therefore, the design key is to locate the most probable failure surface and to construct the by-product layers in a manner that would prevent the critical slip from occurring.

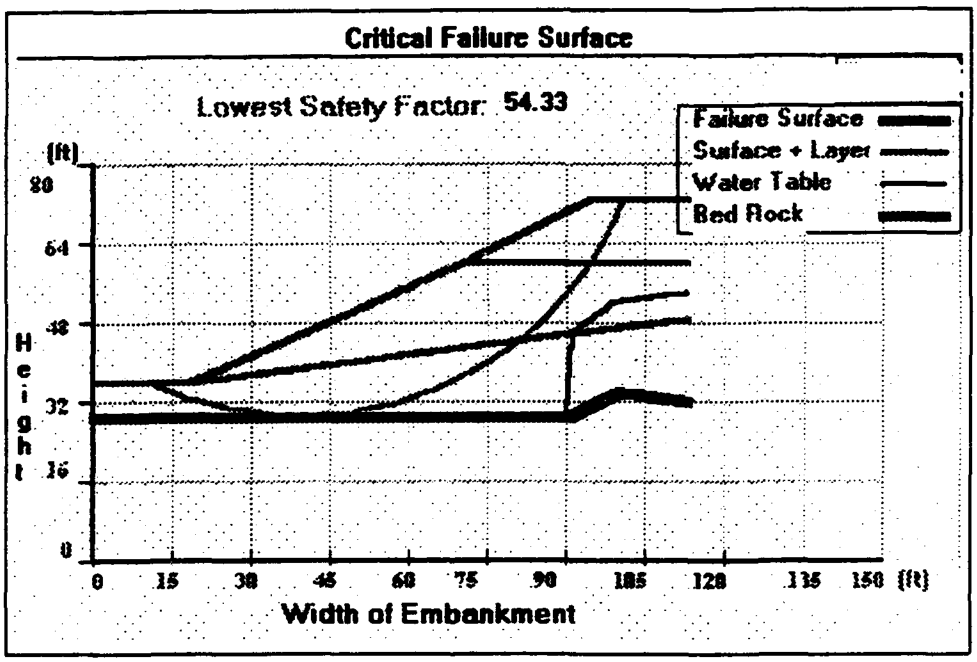

Figure 3.12. SR 541 Embankment, FGD By-Product Reinforcement, Full Depth.

3.3.3 Summary of Scenarios

The eight tests that were conducted using the slope stability routines included in Kim's (1994) IDSSHED system provided some interesting results. The most important result obtained is that slope stability improvements achieved as a results of the FGD by-product buttress system can not be overstated. Though the replaced fill in sub-section 3.3.2.3 yielded a calculated factor of safety of two, it probably would have excavation and fill acquisition costs higher than those associated with the more stable FGD material. The amount of fill required to achieve the calculated safety factor of two greatly exceeds the amount of FGD material required to yield a much higher factor of safety. With the FGD buttress system, the calculated probability of another moderate to deep rotational slide is very small. With a stiff upper fill layer, as discussed in Section 3.3.2.6, the chance of a surface landslide are also greatly reduced. This analysis has clearly shown the benefits of the FGD by-product reinforced embankment with respect to slope stability.

An issue that must be addressed is how the overall embankment stability is a function of the material that covers the top FGD material layer. Since the FGD by-product buttress systems forces all potential failure surfaces out of the embankment, any potential slide will occur in the upper most layer. Assigning the material properties of a cohesionless soil to this layer (Section 3.3.2.6), it can be seen that the calculated factor of safety is merely 1.6. Though this calculated value is low compared to that of the cohesive soil, it further supports the contention that

FGD material serves as an excellent slope stabilizer. The buttress system pushes the critical failure surfaces to the top of the embankment, where they ultimately become a function of the upper most soil's engineering properties. The cohesionless soil produces in a very shallow slide, which can be easily remedied with the planting of deep rooted vegetation or the use of commercially available geotextiles. Alternatively, the FGD itself can be mixed with the natural soil to improve the soils' engineering properties. The Agronomy Department at The Ohio State University has shown that moderate amounts of FGD material can be mixed with soil without negatively effecting the soils' organic qualities necessary to grow surface vegetation (Bigham et al., 1993).

However, if the borrow material is given the properties of a cohesive clay (Section 3.3.2.6), then the overall calculated factor of safety increases significantly. Though this material is quite strong, it is still the FGD by-product buttress that forces the failure surfaces to the top of the embankment. Regardless of the strength properties used for the material that brought the embankment back to its original elevation, the contribution by the buttress system is unquestionable. For input purposes, this layer was assigned the material properties of the original embankment soil for most of the scenarios.

As previously mentioned, the exact geometry of the layers and of the embankment, and the precise elevation of the water table are approximated, but they are within a few meters of what is being used as the model for analysis. Considering the resultant factor of safety is substantially higher than one, it may be inferred that the

dimensions used in this model are well within an acceptable range for necessary engineering accuracy. The results reinforce the point made in the Project Report section that extreme accuracy is not required when working with this material. It has excellent strength properties and workability and is suited for field modifications.

Once again, the importance of this study is to show how significantly the FGD by-product buttress retards moderate to deep embankment slides. Soil, by its very nature, has a high degree of variability associated with its strength parameters when exposed to natural element and forces. Standard embankment construction does not employ laboratory-like precision. For these reasons, it can be inferred that the soil strength values used for this model are at the very least adequate and that the embankment geometry equally representative. It is important to reiterate that with a factor of safety this high, the input parameters, which are based on a combination of laboratory results and good engineering judgment, are truly sufficient.

CHAPTER IV.

LONG-TERM MONITORING

4.1 Background

The types of monitoring equipment that exist today could alone be the topic of a thesis in geotechnical engineering. All available instruments can best be assigned to two general categories. The equipment can either be used for in situ determination of material properties (e.g. strength, permeability, compressibility) or they can be used to monitor performance (e.g. groundwater pressure, deformation, strain) The primary concern of this section is long term stability of the reconstructed embankment and the methods available to accurately monitor its performance.

The need to monitor the performance of a slope is directly proportional to the number of variables associated with the design. The SR 541 embankment is supported by a yet untested buttress of clean coal technology by-product mixture and is infiltrated by several active aquifers. Additionally, the embankment has a history of mass movement and several different soils were utilized in the

reconstruction process. All of the above factors increase the importance of monitoring this slope for horizontal, vertical, and rotational movement, as well as monitoring the groundwater and pore pressure levels.

Ideally, instrumentation could have been used to provide input to the initial design. This proved to be nearly impossible as real world demands dictated the expedited reconstruction schedule. Though no instrumentation was used to determine optimum buttress dimensions or geometry, the field crew's expertise in embankment stabilization problems proved to be quite sufficient. It can be argued that fact-finding was conducted in this "semi-crisis situation" by identifying the precise locations of the aquifers and implementing the means to release them. Additionally, in situ tests were performed to determine certain soil parameters. The systematic excavation of the embankment and subsequent rebuilding of the slope did not call for any active monitoring devices. The procedure was well monitored by site supervisors and utilized trade practices common in routine embankment repair and construction projects.

The need for monitoring becomes paramount at the completion of the project. The initial step is to determine exactly what geotechnical questions must be answered. Every instrument chosen and installed must assist in answering a specific question. The primary parameter of interest in the SR 541 embankment is deformation. The measurable deformation is the effect of the problem, but the cause of the problem may be groundwater conditions. By monitoring both cause and effect, a

relationship between the two events can be established and actions can be taken to remove (or minimize) the cause of the problem.

Prior to the installation of any equipment, predictions must be made to establish instrument ranges and set instrument sensitivities. Estimating maximum possible value leads to a selection of instrument range, whereas the minimum possible value of interest leads to the selection of instrument sensitivity or accuracy. For safety purposes, movements of a predetermined degree may be programmed to activate a warning device. Several warning levels may be developed, each with certain criteria and a respective action. The remedial actions, as well as all other tasks associated with design, construction, and operations needs to be assigned to a certain individual. An example of task assignment for owner-initiated monitoring programs from Dunnicliff is presented as Table 4.1.

Table 4.1. Task Assignments For Monitoring Program.

TASK	ODOT	OSU
Plan monitoring program	*	*
Procure instruments and make factory calibrations		*
Install instruments	*	*
Maintain and calibrate instruments	*	*
Establish and update data collection schedule		*
Collect data		*
Process and present data		*
Interpret and report data		*
Decide on implementation of results	*	

Table 4.1 is tailored to this project, with ODOT as the owner and construction contractor and The Ohio State University as the design consultant and instrumentation specialist.

The instruments that are ultimately chosen to monitor the embankment should have reliability as their overriding characteristic. Simplicity, performance record, and economic efficiency are also considerations. The reliability of the instrument is only as good as the location chosen for instrument installation. Finite element analysis or limit equilibrium studies can be helpful in locating the critical location and most advantageous instrument orientations. The slides predicted in Section 3.3.2.4 can be considered a good starting point. It is recommended that zones of concern be identified, which would include structurally weak areas and zones of high pore water pressure, and the appropriate instruments should be installed there. The selected zones should be representative of the entire cross-section, both in geology and in geometry. There should be a redundant set of instruments, usually of differing operating principles, to confirm the values gathered from the primary group. This redundancy becomes even more important on projects of high cost and/or potential catastrophic failure (Dunnicliff, 1988).

The SR 541 embankment repair effort was neither expensive nor is its potential failure categorically catastrophic, but the success of this project could be far reaching. Therefore, it seems that quality instrumentation and instrument

redundancy would not be unreasonable for the future monitoring of this project. If it can be shown that this embankment that has a history of mass movement remains completely stationary with the FGD buttress system and associated drainage, then this project must be considered a complete success. Such success may lead to the future use of clean coal technology by-products as embankment and roadway stabilizers.

There are many categories of instruments for measuring deformation. An abbreviated list of these categories is shown as Table 4.2.

Table 4.2. Categories of Instrumentation.

	Type of Measured Deformation					
CATEGORY	**HD**	**VD**	**AD**	**RD**	**SD**	**SSD**
Surveying Methods	*	*	*		*	
Surface Extensometers	*	*	*		*	
Tiltmeters				*	*	*
Probe Extensometers	*	*	*			*
Fixed Embankment Extensometers	*	*	*			*
Inclinometers	*	*	*	*		*
Transverse Deformation Gauges	*	*	*			*

HD-Horizontal Deformation VD-Vertical Deformation AD-Axial Deformation
RD-Rotational Deformation SD-Surface Deformation SSD-Subsurface Deformation

The types of instruments that deal mainly with subsurface deformations are of interest.

4.2 Extensometers

Probe extensometers are devices used to measure the distance between points along a common axis. A probe is passed through a pipe and measuring points are identified either mechanically or electrically and the distance between the points is determined by measurements of probe position. The pipe may be vertical, horizontal, or inclined. One of the measuring points must remain fixed, as the movement of the other points must be relative to some datum. This type of device could be used to measure both vertical deformation and compression within an embankment. See Figure 4.1.

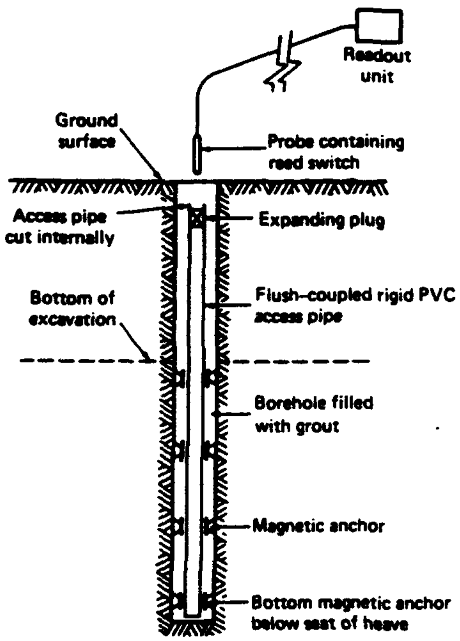

Figure 4.1. Extensometer (After Dunnicliff, 1988).

4.3 Inclinometer

Inclinometers are devices used for monitoring deformation normal to the axis of a pipe by means of a probe passing along the pipe. The probe contains a gravity sensing transducer to measure inclination relative to the vertical plane. The pipes may be installed directly into the fill or into a borehole and are generally placed as close to vertical as possible. For our purposes, the main information available from these instruments is the rate and extent of horizontal embankment movement. With this capability, the potential for landslide movement could be evaluated.

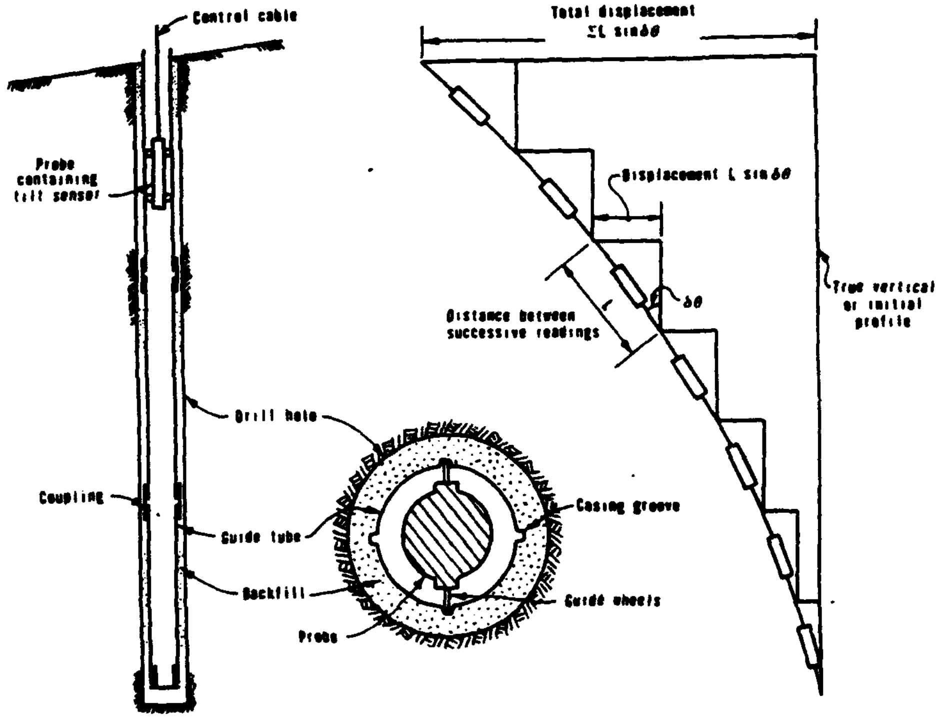

Figure 4-2. Inclinometer (After Schuster and Krizek, 1978).

There are numerous types of inclinometers in use, each possessing certain advantages, limitations and varying degrees of precision. An inclinometer with a force balance accelerometer transducer seems to best fit the parameters of the SR 541 embankment problem. This type of system is the most widely used and is capable of producing digital readouts that are directly usable. Figure 4.2 depicts the principles of the inclinometer operation. The advantages of this system include its long successful experience record and its wide use, and the availability of automatic readout, recording and plotting. Additionally, it is relatively cost efficient and has approximate precision of $\pm$ 1 - 13 mm in 30 meters.

4.4 Deformation Gauges

Transverse deformation gauges are devices used to monitor deformation normal to the axis of a pipe or borehole in which they are installed. Inclinometers, which were previously addressed, are a particular type of deformation gauge. Deformation gauges are particularly well suited for determining the depth and extent of sliding zones in slopes and at measuring the pattern of horizontal deformation within embankments. Other types of deformation gauges include shear plane indicators, inverted pendulums, and plume devices.

The deformation gauge can be as simple as rupture stakes and as complicated as portable borehole deflectometers. Neither extreme is recommended for this embankment, as the shear stakes will not be prudent where the shear plane is some 10 meters to 15 meters below the ground surface and the portable borehole

deflectometer will yield values that will not be more accurate than less costly methods. A suitable deformation gauge could be the shear strip, which consists of a parallel electrical circuit made up of resistors that are mounted on a brittle waterproofed backing strip. As shown in Figure 4.3, the locations of up to two breaks in the strip are determined by measuring resistance at the top and bottom of the strip. A borehole would be drilled into the embankment and a PVC pipe would be installed. Then the shear strip would be inserted with a polyethylene grout tube, grouting with a cement grout, and withdrawing the grout tube. The strip can be installed to an automatic recording system.

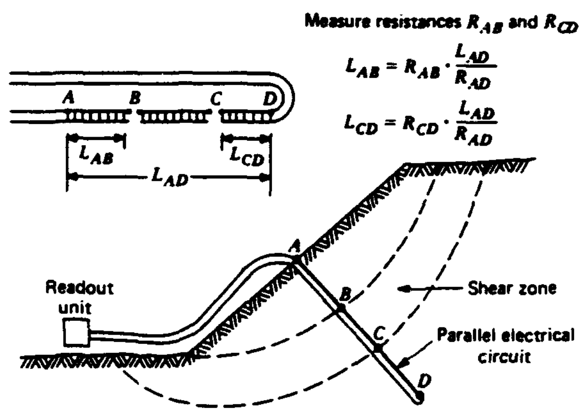

Figure 4.3. Deformation Gauge (After Dunnicliff, 1988).

Another type of deformation gauge is a slip indicator, which provides an economic method of determining the zone of soil mass movement. A flexible PVC tube with a base plate is inserted to the base of a borehole and the region filled with sand.

An indicator probe, attached to a piece of rope, is next lowered to the base of this tube. Where lateral differential slip occurs, the tube will deform and thus indicate the zone of movement. By raising the lower indicator probe, and by lowering a similar probe from the ground level, the exact location of the slip may be determined. A very basic deformation gauge, as shown in Figure 4.4, should be used to monitor any surface movement. Though crude in comparison to other methods, the graduated scale will provide quick and accurate assessments of surfical embankment movement.

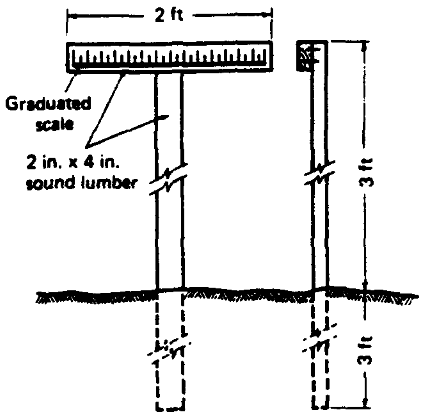

Figure 4.4. Surface Deformation Gauge (After Dunnicliff, 1988).

4.5 Piezometers and Observation Wells

Pore pressure and groundwater level in the slide area are important factors that can best be monitored with commercially available piezometers. An open standpipe (Figure 4.5a) piezometer is merely an observation well in which changes in the

water level can be directly measured by a probe, but is not well suited for impervious soils and partially saturated soils (Schuster and Krizek, 1978). Durability and simplicity are its main advantages and it could be used if economics become a concern in the monitoring of the embankment. A Casagrande type piezometer, shown as Figure 4.5b, has been successfully used in many different materials and is particularly well suited for long-term monitoring. It, too, is characterized by reliability and ease of use. Pneumatic and electric piezometers offer even greater accuracy and often simplicity, but generally have higher costs associated with them.

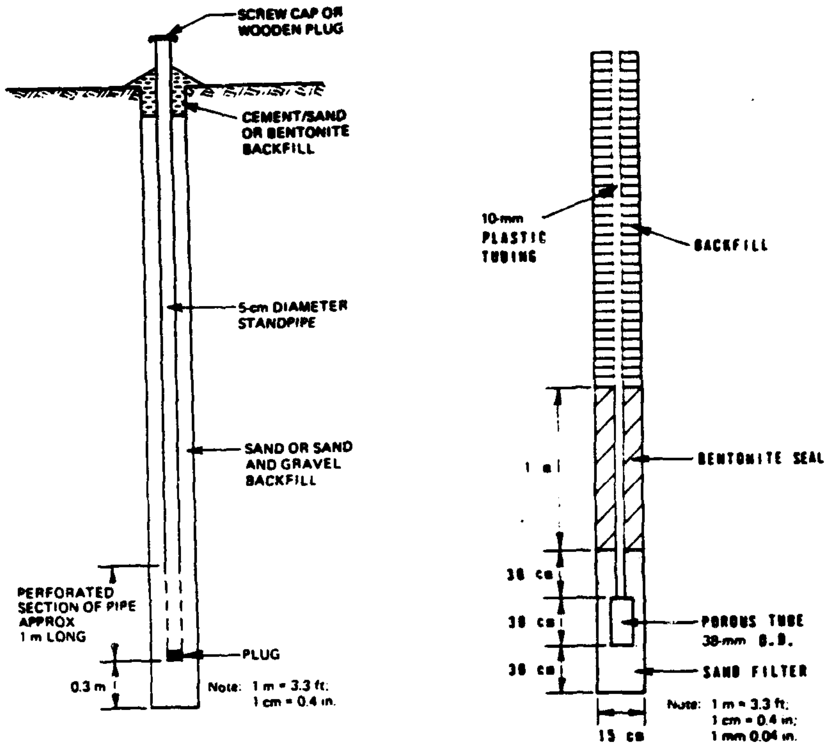

Figure 4.5. Piezometers a) Open-Standpipe b) Casagrande Borehole (After Schuster and Krizek, 1978).

4.6 Recommendations and Conclusion

4.6.1 Recommendations

With advances in geotechnical instrumentation, the monitoring of this embankment should be a relatively simple task. Keeping three factors in mind, cost, reliability, and ease of installation/operation, the author recommends the monitoring configuration that is depicted in Figure 4.6. The plan calls for eight locations to be instrumented, each with an inclinometer, a Casagrande piezometer, and a deformation measuring gauge. The deformation gauges will be some type of bench marking device and will measure surface movement and changes in alignment. The piezometers in the embankment (positions #1 - #6) will measure the water level and pore pressure of the soil. These values will be compared against the readings from the piezometers in the FGD buttress (positions #7 and #8) to determine what level of infiltration is occurring. The inclinometers will measure any horizontal movement or slip of the buttress and the surrounding soil. A single extensometer will be placed at position #9 to measure any vertical deformation or consolidation in the reinforced embankment system.

4.6.2 Conclusions

The previous section is merely the author's recommendation for how the embankment could be instrumented. Modifications will undoubtedly be made as financial conditions change and the value of the overall demonstration project

either increases or decreases. The importance of proper geotechnical instrumentation for monitoring the movement of this embankment is obvious. The question is not whether it should be done, but rather who will fund it and who will actually instrument the embankment. Several methods of instrumentation and monitoring are outlined herein, but full texts exist that offer comprehensive guidance into the field (i.e. Dunnicliff, 1988).

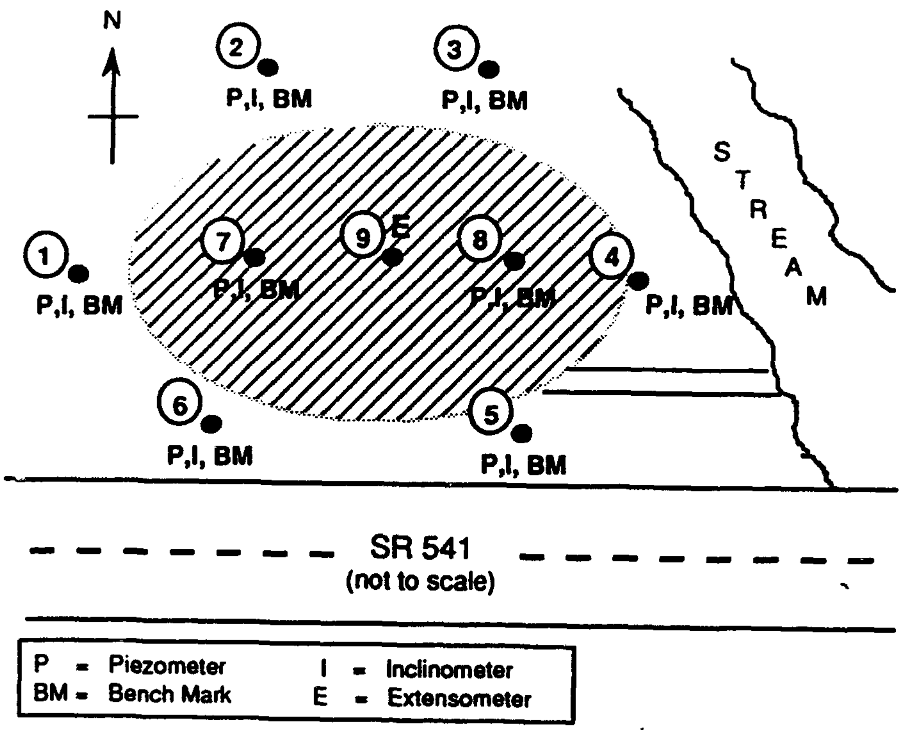

Figure 4.6. Recommended Monitoring of SR 541 Embankment.

CHAPTER V.

DISCUSSION AND CONCLUSIONS

5.1 Discussion

Designing an FGD by-product reinforced embankment requires that some inherent difficulties be addressed. As previously mentioned, the FGD material has a relatively wide range of strength properties that could significantly effect the design geometry. Additionally, there are difficulties associated with the use of the material raising environmental concerns and the normal difficulties encountered in standard fill embankment design. Whereas standard embankment design can be checked against previously constructed structures, the FGD by-product embankment may be the first of its kind (Maher, 1991). The expertise in this field is very limited and significant strides will not be seen until mistakes are made and learned from.

It is undisputed that the effort to clean up the atmospheric pollution, which has been associated with the burning of coal, has produced a new and ever increasing solid waste problem. Estimates show that in U.S. power plants alone, over 18.1

million metric tons of this waste are being produced annually (Taha, 1993). This number is expected to double (35-40 million metric tons) as the provisions of the 1990 Clean Air amendment take full effect (Taha, 1993). The Department of Energy estimates that the amount of solid waste generated over the life of one 500 MW power plant would fill a 200 hectare disposal pond to a depth of 12.2 meters (U.S. Department of Energy, 1992). Numbers of this magnitude justify efforts to find alternatives to land filling the waste associated with the FGD process.

5.2 Conclusions

With successful instrumentation and proper monitoring, the SR 541 Embankment repair stands to be a showcase demonstration project. Despite the lack of knowledge associated with working with FGD material and the harsh winter weather, the ODOT repair crew performed outstandingly in the reconstruction effort. The systematic excavation, drain and buttress installation, and subsequent embankment reconstruction was recorded on film and chronicled herein, and can be referenced for future projects of similar nature. Low cost and ease of construction have already been proven, leaving long-term stability and environmental impact as the only unanswered questions. Appropriate monitoring, as outlined in Section 4.6.1, should answer these remaining questions. If what has been observed so far, with respect to environmental impact and the Tidd material strength, is indicative of future results, then this demonstration project will surely be considered a success. The beneficiaries of this successful demonstration project are ODOT, AEP, and the public.

It has been shown how dry FGD by-products can be successfully incorporated in numerous constructive applications. The demonstration projects that have been completed, the PFBC by-product feed lots and embankment, and the ramp constructed of spray dryer material, are all performing up to expectations. Though early in their design lives, none of these projects are exhibiting any signs of structural failure. All of these projects have been typified by simplicity of construction and have proven that no special equipment or training is necessary. Results reinforce the point that extreme precision is not required when working with this material and its excellent strength properties and workability are suited for field modifications. Other demonstration projects are in the early implementation or planning stages, so it would be premature to draw any substantive conclusions. It is hoped that the visibility and the success of these projects lead to the increased acceptance of FGD waste as a viable construction material as further, and more creative uses, for this product are developed.

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Bishop Method of Slices - No Friction Angle

Slice	b (ft	h (ft	Density	Wgt(lb/ft	a (deg)	a (rad)	Wsinalpha	u	ub	cb	phi(rad)	Equa 1	Equa 2	F=1.5	m
1	8	8	120	7680	55	0.9599	6291.0877	384	3072	4800	0	0	4800	1.7434	8368.5
2	8	18	120	17280	44	0.7679	12003.697	864	6912	4800	0	0	4800	1.3902	6672.8
3	8	22	120	21120	36	0.6283	12414.025	1056	8448	4800	0	0	4800	1.2361	5933.1
4	8	24	120	23040	28	0.4887	10816.625	1152	9216	4800	0	0	4800	1.1326	5436.3
5	8	24	120	23040	21	0.3665	8256.7976	1152	9216	4800	0	0	4800	1.0711	5141.5
6	8	23	120	22080	16	0.2793	6086.0728	1104	8832	4800	0	0	4800	1.0403	4993.4
7	8	21	120	20160	10	0.1745	3500.7473	1008	8064	4800	0	0	4800	1.0154	4874
8	8	16	120	15360	6	0.1047	1605.5572	768	6144	4800	0	0	4800	1.0055	4826.4
9	8	11	120	10560	0	0	0	528	4224	4800	0	0	4800	1	4800
10	8	5	120	4800	-6	-0.105	-501.7366	240	1920	4800	0	0	4800	1.0055	4826.4
						Sum	60472.872								55873

0.9239
Ans.

Bishop Method of Slices - With Friction Angle

Sl.	(ft	(ft	Density	Wgt(lb/ft	a,deg	a (rad)	Wsinalpha	u	ub	cb	phi(rad)	Equa 1	Equa 2	F=1.3	m	F=1.5	m	F=1.75	m
1	8	8	120	7680	55	0.9599	6291.0877	384	3072	6400	0.3491	1677.17	8077.17	1.245	10060	1.2948	10458	1.3442	10857
2	8	18	120	17280	44	0.7679	12003.697	864	6912	6400	0.3491	3773.64	10173.6	1.094	11133	1.1263	11458	1.1577	11778
3	8	22	120	21120	36	0.6283	12414.025	1056	8448	6400	0.3491	4612.23	11012.2	1.027	11311	1.0508	11572	1.0738	11825
4	8	24	120	23040	28	0.4887	10816.625	1152	9216	6400	0.3491	5031.52	11431.5	0.986	11269	1.0031	11467	1.0198	11658
5	8	24	120	23040	21	0.3665	8256.7976	1152	9216	6400	0.3491	5031.52	11431.5	0.967	11057	0.9799	11201	0.992	11340
6	8	23	120	22080	16	0.2793	6086.0728	1104	8832	6400	0.3491	4821.88	11221.9	0.963	10807	0.9728	10915	0.9817	11017
7	8	21	120	20160	10	0.1745	3500.7473	1008	8064	6400	0.3491	4402.58	10802.6	0.968	10453	0.9738	10519	0.9795	10581
8	8	16	120	15360	6	0.1047	1605.5572	768	6144	6400	0.3491	3354.35	9754.35	0.977	9527.7	0.9805	9564.2	0.984	9598.3
9	8	11	120	10560	0	0	0	528	4224	6400	0.3491	2306.12	8706.12	1	8706.1	1	8706.1	1	8706.1
10	8	5	120	4800	-6	-0.105	-501.7366	240	1920	6400	0.3491	1048.23	7448.23	1.036	7716.3	1.0316	7685.3	1.028	7656.6
						Sum	60472.872							Sum	102040		103546		105016
														F.S.	1.6874		1.7123		1.7366
																			Answer

PROFIL
Slope Stability Analysis with Drain, Saturated Condition
4 3
0.0 36. 18. 36. 2
18. 36. 94. 73. 1
94. 73. 113. 73. 1
18. 36. 113. 49. 2
SOIL
2
116.4 130. 800. 20. 0.0 0.0 1
116.4 130. 400. 0. 0.0 0.0 1
WATER
1 62.4
7
0.0 29.
30. 29.
58. 29.
90. 29.
91. 46.
99. 52.
113. 54.
LIMITS
6 6
0.0 29. 30. 29.
30. 29. 58. 29.
58. 29. 71. 29.
71. 29. 90. 29.
90. 29. 100. 34.
100. 34. 113. 32.
CIRCL2
10 10
0.0 94. 94. 113.
0.0 8. 0.0 0.0

```
PROFIL
Slope Stability Analysis with Drained Condition
4 3
0.0 36. 18. 36. 2
18. 36. 94. 73. 1
94. 73. 113. 73. 1
18. 36. 113. 49. 2
SOIL
2
116.4 130. 800. 20. 0.0 0.0 1
116.4 130. 400. 20. 0.0 0.0 1
WATER
1 62.4
7
0.0 29.
30. 29.
58. 29.
90. 29.
91. 46.
99. 52.
113. 54.
LIMITS
6 6
0.0 29. 30. 29.
30. 29. 58. 29.
58. 29. 71. 29.
71. 29. 90. 29.
90. 29. 100. 34.
100. 34. 113. 32.
CIRCL2
10 10
0.0 94. 94. 113.
0.0 8. 0.0 0.0
```

PROFIL
Slope Stability Analysis with FGD, Specific Failure


```
12 3
0.0 36. 18. 36. 4
18. 36. 94. 73. 2
94. 73. 113. 73. 2
74. 60. 108. 60. 1
108. 60. 109. 56. 1
73. 56. 74. 60. 1
73. 56. 109. 56. 3
89. 45. 113. 49. 4
59. 45. 89. 45. 1
89. 45. 90. 29. 1
18. 36. 59. 45. 4
58. 29. 59. 45. 1
SOIL
4
128.4 128.4 94741. 35. 0.0 0.0 1
115. 135. 1200. 29. 0.0 0.0 1
116.4 130. 800. 20. 0.0 0.0 1
116.4 130. 400. 0. 0.0 0.0 1
WATER
1 62.4
7
0.0 29.
30. 29.
58. 29.
90. 29.
91. 46.
99. 52.
113. 54.
LIMITS
6 6
0.0 29. 30. 29.
30. 29. 58. 29.
58. 29. 71. 29.
71. 29. 90. 29.
90. 29. 100. 34.
100. 34. 113. 32.
SURBIS
8
12. 36. 30. 29. 45. 30. 60. 32.
72. 37. 80. 44. 88. 54. 95. 73.
EXECUT
```

```
PROFIL
 Ohio Route 541 Embankment Design
15 4
0.0 36.0 18.0 36.0 4
18.0 36.0 66.0 60.0 3
66.0 60.0 94.0 74.0 2
94.0 74.0 113.0 74.0 2
66.0 60.0 74.0 60.0 3
74.0 60.0 108.0 60.0 1
108.0 60.0 113.0 60.0 3
108.0 60.0 109.0 56.0 1
73.0 56.0 74.0 60.0 1
73.0 56.0 109.0 56.0 3
89.0 45.0 113.0 49.0 4
59.0 45.0 89.0 45.0 1
89.0 45.0 90.0 29.0 1
18.0 36.0 59.0 45.0 4
58.0 29.0 59.0 45.0 1
SOIL
4
128.4 128.4 94741. 0. 0.0 0.0 1
120.0 120.0 800.0 20.0 0.0 0.0 1
120.0 120.0 800.0 20.0 0.0 0.0 1
120. 130. 800. 0. 0.0 0.0 1
WATER
1 62.4
7   0.0 29.0
45.0 29.0
60.0 29.0
90.0 29.0
91.0 46.0
99.0 52.0
113.0 54.0
LIMITS
6 6
0.0 29.0 30.0 29.0
30.0 29.0 58.0 29.0
58.0 29.0 71.0 29.0
71.0 29.0 90.0 29.0
90.0 29.0 100.0 34.0
100.0 34.0 113.0 32.0
CIRCL2
10 10
0.0 94.0 94.0 113.0
0.0 8.0 0.0 0.0
```

```
PROFIL
 Ohio Route 541 Embankment Design
15 4
0.0 36.0 18.0 36.0 4
18.0 36.0 66.0 60.0 3
66.0 60.0 94.0 74.0 2
94.0 74.0 113.0 74.0 2
66.0 60.0 74.0 60.0 3
74.0 60.0 108.0 60.0 1
108.0 60.0 113.0 60.0 3
108.0 60.0 109.0 56.0 1
73.0 56.0 74.0 60.0 1
73.0 56.0 109.0 56.0 3
89.0 45.0 113.0 49.0 4
59.0 45.0 89.0 45.0 1
89.0 45.0 90.0 29.0 1
18.0 36.0 59.0 45.0 4
58.0 29.0 59.0 45.0 1
SOIL
4
128.4 128.4 94741. 0. 0.0 0.0 1
120.0 120.0 800.0 20.0 0.0 0.0 1
120.0 120.0 800.0 20.0 0.0 0.0 1
120. 130. 800. 0. 0.0 0.0 1
WATER
1 62.4
7   0.0 29.0
15. 32.
30. 34.
59. 46.
76. 48.
99. 52.0
113.0 54.0
LIMITS
6 6
0.0 29.0 30.0 29.0
30.0 29.0 58.0 29.0
58.0 29.0 71.0 29.0
71.0 29.0 90.0 29.0
90.0 29.0 100.0 34.0
100.0 34.0 113.0 32.0
CIRCL2
10 10
0.0 94.0 94.0 113.0
0.0 8.0 0.0 0.0
```

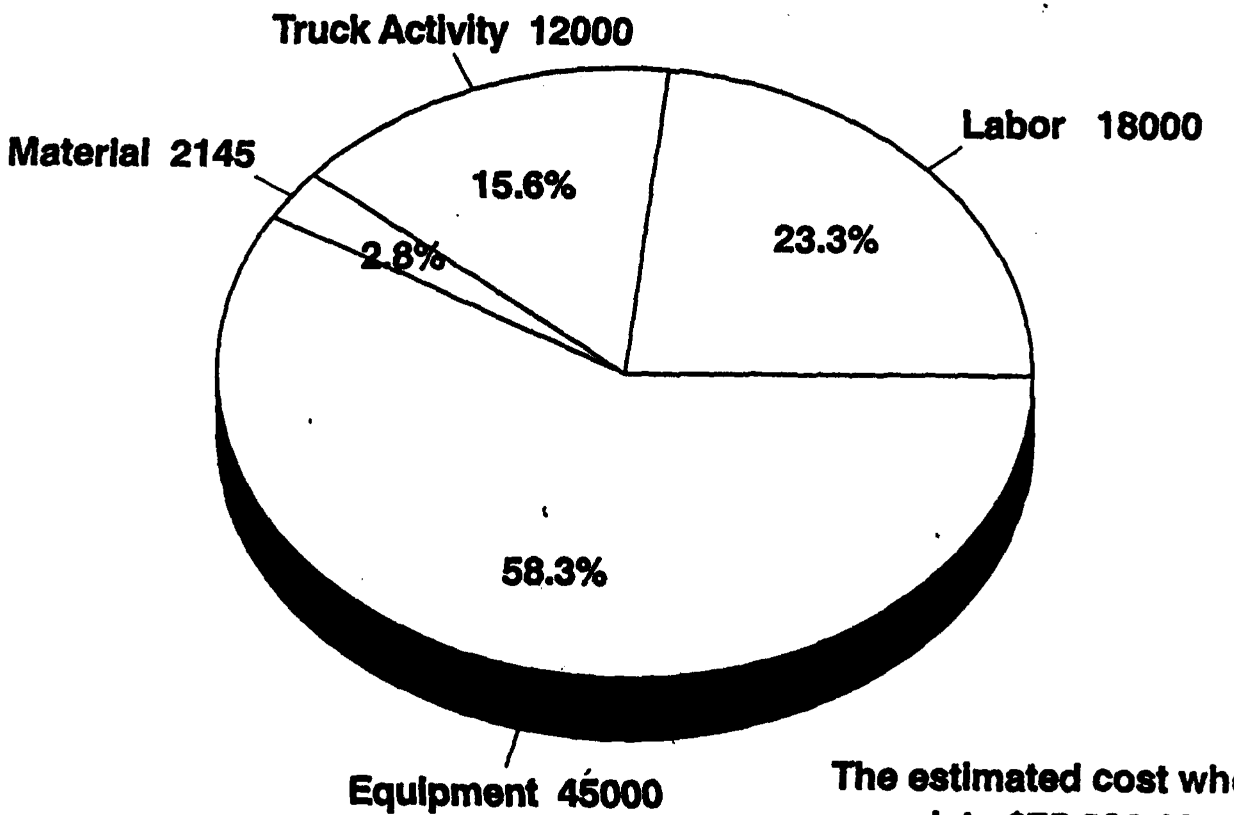
PROFIL
Ohio Route 541 Embankment Design
15 4
0.0 36.0 18.0 36.0 4
18.0 36.0 66.0 60.0 3
66.0 60.0 94.0 74.0 2
94.0 74.0 113.0 74.0 2
66.0 60.0 74.0 60.0 3
74.0 60.0 108.0 60.0 1
108.0 60.0 113.0 60.0 3
108.0 60.0 109.0 56.0 1
73.0 56.0 74.0 60.0 1
73.0 56.0 109.0 56.0 3
89.0 45.0 113.0 49.0 4
59.0 45.0 89.0 45.0 1
89.0 45.0 90.0 29.0 1
18.0 36.0 59.0 45.0 4
58.0 29.0 59.0 45.0 1
SOIL
4
128.4 128.4 94741. 0. 0.0 0.0 1
120. 120. 800. 0. 0.0 0.0 1
120. 120. 800. 20. 0.0 0.0 1
120. 130. 800. 0. 0.0 0.0 1
WATER
1 62.4
7 0.0 29.0
45.0 29.0
60.0 29.0
90.0 29.0
91.0 46.0
99.0 52.0
113.0 54.0
LIMITS
6 6
0.0 29.0 30.0 29.0
30.0 29.0 58.0 29.0
58.0 29.0 71.0 29.0
71.0 29.0 90.0 29.0
90.0 29.0 100.0 34.0
100.0 34.0 113.0 32.0
CIRCL2
10 10
0.0 94.0 94.0 113.0
0.0 8.0 0.0 0.0

PROFIL
Slope Stability Analysis with Shale
6 4
0.0 36. 18. 36. 3
18. 36. 70. 60. 2
70. 60. 94. 73. 1
94. 73. 113. 73. 1
70. 60. 113. 60. 2
18. 36. 113. 49. 3
SOIL
3
115. 135. 1200. 29. 0.0 0.0 1
116.4 130. 800. 20. 0.0 0.0 1
116.4 130. 400. 0. 0.0 0.0 1
WATER
1 62.4
7
0.0 29.
30. 29.
58. 29.
90. 29.
91. 46.
99. 52.
113. 54.
LIMITS
6 6
0.0 29. 30. 29.
30. 29. 58. 29.
58. 29. 71. 29.
71. 29. 90. 29.
90. 29. 100. 34.
100. 34. 113. 32.
CIRCL2
10 10
0.0 94. 94. 113.
0.0 8. 0.0 0.0

PROFIL
Slope Stability Analysis with Full Depth FGD By-product
6 4
0.0 36. 18. 36. 3
18. 36. 70. 60. 2
70. 60. 94. 73. 1
94. 73. 113. 73. 1
70. 60. 113. 60. 2
18. 36. 113. 49. 3
SOIL
3
128. 128. 94741. 35. 0.0 0.0 1
128. 128. 94741. 35. 0.0 0.0 1
116.4 130. 400. 0. 0.0 0.0 1
WATER
1 62.4
7
0.0 29.
30. 29.
58. 29.
90. 29.
91. 46.
99. 52.
113. 54.
LIMITS
6 6
0.0 29. 30. 29.
30. 29. 58. 29.
58. 29. 71. 29.
71. 29. 90. 29.
90. 29. 100. 34.
100. 34. 113. 32.
CIRCL2
10 10
0.0 94. 94. 113.
0.0 8. 0.0 0.0

Slip Cost Profile S.R.541--18.1 M/Mark

Coshocton County



The estimated cost when complete:$75,000.00